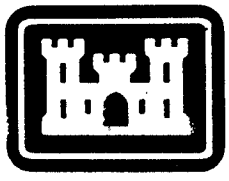
ENGINEER MANUAL

EM 1110-3-162
April 1984

ENGINEERING AND DESIGN

Water Supply, Water Treatment

Mobilization Construction



DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
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DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, D.C. 20314

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Engineer Manual
No. 1110-3-162

9 April 1984

Engineering and Design
WATER SUPPLY, WATER TREATMENT
Mobilization Construction

1. Purpose. This manual establishes water quality standards and design criteria for water treatment processes at U.S. Army mobilization facilities.

2. Applicability. This manual is applicable to all field operating activities having mobilization construction responsibilities.

3. Discussion. Criteria and standards presented herein apply to construction considered crucial to a mobilization effort. These requirements may be altered when necessary to satisfy special conditions on the basis of good engineering practice consistent with the nature of the construction. Design and construction of mobilization facilities must be completed within 180 days from the date notice to proceed is given with the projected life expectancy of five years. Hence, rapid construction of a facility should be reflected in its design. Time-consuming methods and procedures, normally preferred over quicker methods for better quality, should be de-emphasized. Lesser grade materials should be substituted for higher grade materials when the lesser grade materials would provide satisfactory service and when use of higher grade materials would extend construction time. Work items not immediately necessary for the adequate functioning of the facility should be deferred until such time as they can be completed without delaying the mobilization effort.

FOR THE COMMANDER:

PAUL F. KAVANAUGH
Colonel, Corps of Engineers
Chief of Staff

DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314

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CHAPTER 1

GENERAL

1-1. Purpose and scope. This manual, intended for planners and design engineers, presents information on water quality standards and design criteria for water treatment processes at Army mobilization facilities. This manual also establishes criteria to be followed in determining the necessity for and the extent of treatment, and on procedures applicable to the planning of water treatment projects.

1-2. Consultation with Government agencies. State health department, state water resource, and U.S. Environmental Protection Agency personnel as appropriate, should be consulted in the early stages of project planning regarding supply sources and associated water treatment needs. State agencies have established design guidelines based on local conditions and experiences. Information regarding these guidelines is available from the division of engineering within the state agency responsible for environmental protection. Consultation with state engineers will provide valuable information relative to plant design and water treatment experience in the state or region. It is also advisable to confer with management and operating personnel of nearby water supply utilities.

1-3. Water quality criteria and standards. Information on current criteria and standards for raw and potable water are presented in TB MED 576.

1-4. Definitions. Definitions for water treatment works will be as presented in EM 1110-3-160.

CHAPTER 2

WATER TREATMENT SYSTEMS

2-1. General design criteria.

a. Water treatment plants. Water treatment plants at mobilization facilities must produce water of sufficient quality and quantity for all intended purposes. If the water is to be used for human consumption, it must be free of organisms or substances posing health hazards at all times. The overall water quality objective can be met if the water delivered to service meets the drinking water standards presented in TB MED 576. Typical design examples can be found in appendix A.

b. Water storage and distribution. The quality of water obtained at the user's tap is not determined solely by water treatment operations. Raw water quality and conditions in treated water storage and water distribution systems also affect the quality of tap water. Consequently, protection of raw water quality and finished water storage and delivery systems to the maximum practicable extent is essential. Excellence in water treatment is partially nullified unless other water system components are adequately designed, maintained, and operated. General considerations on water supplies can be found in EM 1110-3-160. Information on water sources is contained in EM 1110-3-161; on water storage in EM 1110-3-163; and on water distribution in EM 1110-3-164.

2-2. Plant siting. The following items will be considered in choosing a plant site.

- Proximity to the source of raw water.
- Proximity to the area to be served.
- Ability to locate intake upstream of wastewater discharges.
- Land availability and costs.
- Potential for flooding of the site.
- Availability and reliability of electric power.
- Geology and topography of the site.
- Availability of transportation facilities.
- Legal obligations or restrictions.

2-3. Process selection and design. The selection and design of the water treatment processes to be used at a particular facility are dictated by practicability, reliability, flexibility, and overall economics. Engineers experienced in water treatment plant design are needed to determine the best treatment system for any particular situation, and their advice should be obtained in the early stages of project planning. Detailed information about major treatment processes is given in chapter 3.

2-4. Reliability.

a. Duplicates of essential items, including pumps, settling basins, flocculators, filters, and chemical feeders should be provided as appropriate to allow periodic maintenance and repair. The degree of importance of each item must be evaluated on a case-by-case basis, considering that safe water has to be supplied at all times. One train of essential items may be constructed initially if this would allow the facility to be operational within the allotted mobilization time. Backup units can be constructed subsequently.

b. Installation of emergency generating facilities at the water treatment plant will be required if lengthy power outages are probable. Likewise, if the delivery of crucial chemical supplies is uncertain, larger than normal stores of these chemicals must be kept on hand, which would necessitate larger than normal chemical storage areas.

2-5. Operating considerations. To simplify plant operations, the following guidelines should be observed during the design stage.

a. Eliminate any unnecessary equipment and operations.

b. Locate operations requiring frequent operator attention reasonably close together. Filters, flocculators, and chemical feeding equipment generally require the most attention.

c. Simplify chemical handling and feeding as much as possible. Unloading and storage areas for chemicals should be easily maintained, readily accessible, and close to the point of application of chemicals.

d. Design flexibility into plants treating surface water to handle raw water quality changes.

2-6. Plant capacity. The water treatment plant will be sized to treat sufficient water to meet the requirements given in EM 1110-3-160. Care should be taken not to underestimate special water demands.

CHAPTER 3

WATER TREATMENT PROCESSES

3-1. Process selection factors. The design of treatment facilities will be determined by considering all engineering, labor, materials, equipment, and energy factors.

3-2. Preliminary treatment processes. Surface waters typically contain fish and debris which can clog or damage pumps, clog pipes, and cause problems in water treatment. Surface waters may also contain high concentrations of suspended matter. Preliminary treatment is employed for removal of debris and part of the suspended materials.

a. Screens.

(1) Coarse screens or racks. Coarse screens, often termed bar screens or racks, must be provided to intercept large, suspended or floating material. Such screens or racks are made of 1/2-inch to 3/4-inch metal bars spaced to provide 1 to 3 inch openings.

(2) Fine screens. Surface waters require screens or strainers for removal of material too small to be intercepted by the coarse rack. These may be basket-type, in-line strainers, manually or hydraulically cleaned by backwashing, or of the traveling type, which are cleaned by water jets. Fine-screen, clear openings should be approximately 3/8 inch. The velocity of the water in the screen openings should be less than 2 fps at maximum design flow through the screen and at minimum screen submergences which occur at minimum anticipated flows.

(3) Ice clogging. In northern areas, screens may be clogged by frazil or anchor ice. Exposure of racks or screens to cold air favors ice formation on submerged parts and should be avoided to the maximum practicable extent. Steam or electric heating, diffusion aeration, and flow reversal have been used to overcome ice problems.

(4) Disposal of screenings. Project planning must include provisions for the disposal of debris removed by coarse and fine screens.

b. Flow measurement. Water treatment processes, such as chemical application, are related to the rate of flow of raw water. Therefore, it is essential that accurate flow-rate measurement equipment is provided. Pressure differential devices of the Venturi type are commonly used for measurement of flow in pressure conduits. An alternative selection for pressure conduits is a magnetic flow meter if the minimum velocity through the meter will be 5 fps or more. A Parshall flume can be used for metering in open channels. Flow signals from the metering device selected should be transmitted to the treatment plant control center.

c. Flow division. While not a treatment process, flow division (flow splitting) is an important treatment plant feature that must be considered at an early stage of design. To insure continuity of operation during major maintenance, plants are frequently designed with parallel, identical chemical mixing, and sedimentation facilities. No rigid rules can be given for the extent of duplication required because a multiplicity of factors influence the decision. Normally, aerators are not provided in duplicate. Presedimentation basins may not require duplication if maintenance can be scheduled during periods of relatively low raw water sediment load or if the following plant units can tolerate a temporary sediment overload. If it is determined that presedimentation at all times is essential for reliable plant operation, then the flow division should be made ahead of the presedimentation basins by means of identical splitting weirs arranged so that flow over either weir may be stopped when necessary. During normal operation, the weirs would accomplish a precise equal division of raw water, regardless of flow rate, to parallel subsequent units: rapid-mix, slow-mix, and sedimentation. The water would then be combined and distributed to the filters. If presedimentation units are not provided, then the flow is commonly split ahead of the rapid-mix units. If flow division is deemed necessary, the design should allow flexibility in construction such that at least one system, or train, of units can be constructed first. Duplicate trains can then be constructed after the first train is in operation.

d. Sand traps. Sand traps are not normally required at surface water treatment plants. Their principal application is for the removal of fine sand from well water. The presence of sand in well water is usually a sign of improper well construction or development. If sand pumping cannot be stopped by reworking the well, the sand must be removed from the well water. Otherwise, it will create serious problems in the distribution system by clogging service pipes, meters, and plumbing. Centrifugal sand separators provide an effective means of sand removal.

e. Plain sedimentation. Plain sedimentation, also termed "presedimentation," is accomplished without the use of coagulating chemicals. Whether plain sedimentation is essential is a judgment decision influenced by the experience of plants treating water from the same source. Water derived from lakes or impounding reservoirs rarely requires presedimentation treatment. On the other hand, water obtained from notably sediment-laden streams, such as those found in parts of the Midwest, require presedimentation facilities for removal of gross sediment load prior to additional treatment. Presedimentation treatment should receive serious consideration for water obtained from rivers whose turbidity value frequently exceeds 1,000 units.

(1) Plain sedimentation basins. Plain sedimentation or presedimentation basins may be square, circular, or rectangular and are equipped with sludge removal mechanisms.

(2) Design criteria. Detention time should be approximately 3 hours. Basin depth should be in the approximate range of 10 to 15 feet, corresponding to upflow rates of 600 to 900 gpd per square foot. Short-circuiting can be minimized by careful attention to design of inlet and outlet arrangements. Weir loading rates should not exceed approximately 20,000 gpd per foot. Where presedimentation treatment is continuously required, duplicate basins should be provided.

3-3. Aeration. The term "aeration" refers to the processes in which water is brought into contact with air for the purpose of transferring volatile substances to or from the water. These volatile substances include oxygen, carbon dioxide, hydrogen sulfide, methane, and unidentified volatile organic compounds responsible for tastes and odor. Aeration is frequently employed at plants designed for iron and manganese removal. The oxygen added by aeration oxidizes dissolved iron and manganese to insoluble forms which can then be removed by sedimentation and filtration.

a. Purposes of aeration. The principal objectives of aeration are:

(1) Addition of oxygen to water for the oxidation of iron, manganese, and hydrogen sulfide and to a limited extent, organic matter.

(2) Partial removal of substances that may interfere with or add to the cost of water treatment. Examples: Removal of hydrogen sulfide prior to chlorination; removal of carbon dioxide prior to lime softening.

(3) Reduction of the concentration of taste-and-odor producing substances such as hydrogen sulfide, and to a limited extent, volatile organic compounds.

b. Types of aerators. Three types of aerators are commonly employed. These are: (1) waterfall aerators exemplified by spray nozzle, cascade, and multiple-tray units, (2) diffusion or bubble aerators which involve passage of bubbles of compressed air through the water, and (3) mechanical aerators employing motor-driven impellers alone or in combination with air injection devices. Of the three types, waterfall aerators, employing multiple trays, are the most frequently used in water treatment. The efficiency of multiple-tray aerators can be increased by the use of enclosures and blowers to provide counterflow ventilation. The use of diffusion or bubble aerators will be discouraged because of the complexity of compressed air equipment.

c. Use of aeration. Aeration will be considered only when treating ground water with high levels of dissolved iron, manganese, hydrogen sulfide, or other dissolved gases and volatile organic compounds which create odor or taste problems. Aeration will be considered for the

removal of carbon dioxide in ground water if lime softening is to be used. A laboratory test to determine the efficiency of aeration is usually required.

3-4. Coagulation and flocculation. Coagulation means a reduction in the forces which tend to keep suspended particles apart. The joining together of small particles into larger, settleable, and filterable particles is flocculation. Thus, coagulation precedes flocculation, and the two processes must be considered conjunctively.

a. Purposes of coagulation and flocculation. Raw water supplies, especially surface water supplies, often contain a wide range of matter, including minerals, organic substances, and microscopic organisms ranging in size from about 0.001 to 1.0 micrometer. Particles in this size range are often referred to as "colloidal" particles. Larger particles, such as sand and silt, readily settle out of water during plain sedimentation, but the settling rate of colloidal particles is so low that removal of colloidal particles by plain sedimentation is not practicable. Chemical coagulation and flocculation processes are required to remove these smaller particles in sedimentation basins.

b. Chemical coagulants. The most frequently used chemical coagulant is aluminum sulfate: $(Al_2(SO_4)_3 \cdot 14H_2O)$, which averages about 17 percent Al_2O_3 and is also called "alum" or "filter alum." Other aluminum compounds used as coagulants are potash alum and sodium aluminate, principally the latter. Other coagulants include ferric sulfate, chlorinated copperas (ferrous sulfate), and ferric chloride. Organic polyelectrolyte compounds, alone or in combination with conventional chemical coagulants, are also employed. The coagulation-flocculation processes are accomplished step-wise by short-time rapid mixing to disperse the chemical coagulant followed by a longer period of slow mixing to complete the flocculation process. Laboratory jar tests will normally be performed to determine the type and quantity of coagulant(s) best suited to the water requiring treatment.

c. Design criteria for mixing. Criteria for rapid- and slow-mix processes have been developed on the basis of detention time, power input, velocity gradient (G) and the product (Gt) of velocity gradient and detention time. The values of G and Gt are computed from the following equations:

$$G = \left(\frac{P}{uV}\right)^{1/2}$$

$$Gt = \frac{1}{Q}\left(\frac{PV}{u}\right)^{1/2}$$

and

where:

Gt = product of G and t, a dimensionless number
G = velocity gradient (fps/foot; or second^{-1})
P = the power dissipated in the water (ft-lb/second)
u = water viscosity (pound$_f$-seconds/square feet) (u = 2.73 x 10^{-5} at 50 degrees F.)
Q = flow through the sedimentation unit (cfs)
V = volume of mixing basin (cubic feet)
t = mixer detention time (seconds)

(1) Rapid mixing. The coagulant is introduced into the water by rapid mixing and coagulation is initiated. Detention periods for rapid-mix units usually range from 10 to 30 seconds with installed mixer power approximately 0.25 to 1.0 hp per mgd. Power and detention time should be matched so that values of G will be in the approximate range: 500 to 1,000/second. A wire-to-water efficiency of 80 percent, a water temperature of 50 degrees F., a power input of 1.0 hp per mgd and a detention time of 10 seconds, yield a G value of about 1,000/second and a Gt value of 10,000. Similarly, a 30-second detention time gives a G value of about 600 and a Gt value of 18,000. Long detention periods for rapid-mix basins should be avoided because of higher power requirements and inferior coagulation results. The rapid-mix basin should be designed to minimize short circuiting.

(2) Slow mix. Flocculation is accomplished by slow mixing. Detention periods should range from 30 minutes to 60 minutes, with installed mixer power of approximately 0.1 to 3.5 hp per mgd. G values in the range of 20/second to 100/second are commonly employed. Corresponding Gt values will, therefore, be in the range of 36,000 to 360,000. Tapered, slow mixing with G decreasing from a maximum of about 90/second down to 50/second and then to 30/second can be used and will generally produce some improvement in flocculation. Somewhat higher G values, up to 200/second, are employed in some water softening plants. For normal flocculation, using alum or iron salts, the maximum peripheral speed of the mixing units should not exceed about 2.0 fps and provisions should be made for speed variation. To control short circuiting, two to three compartments are usually provided. Compartmentation can be achieved by the use of baffles. Turbulence following flocculation must be avoided. Conduits carrying flocculated water to sedimentation basins should be designed to provide velocities of not less than 0.5 fps and not more than 1.5 fps. Weirs produce considerable turbulence and should not be used immediately following flocculation.

3-5. Sedimentation basins. Sedimentation follows flocculation, and the most common types of sedimentation basins in general use are shown in figures 3-1 and 3-2. A minimum of two basins should be provided to allow one unit to be out of service for repair, but construction of the second unit may be delayed until the first unit is operational. The

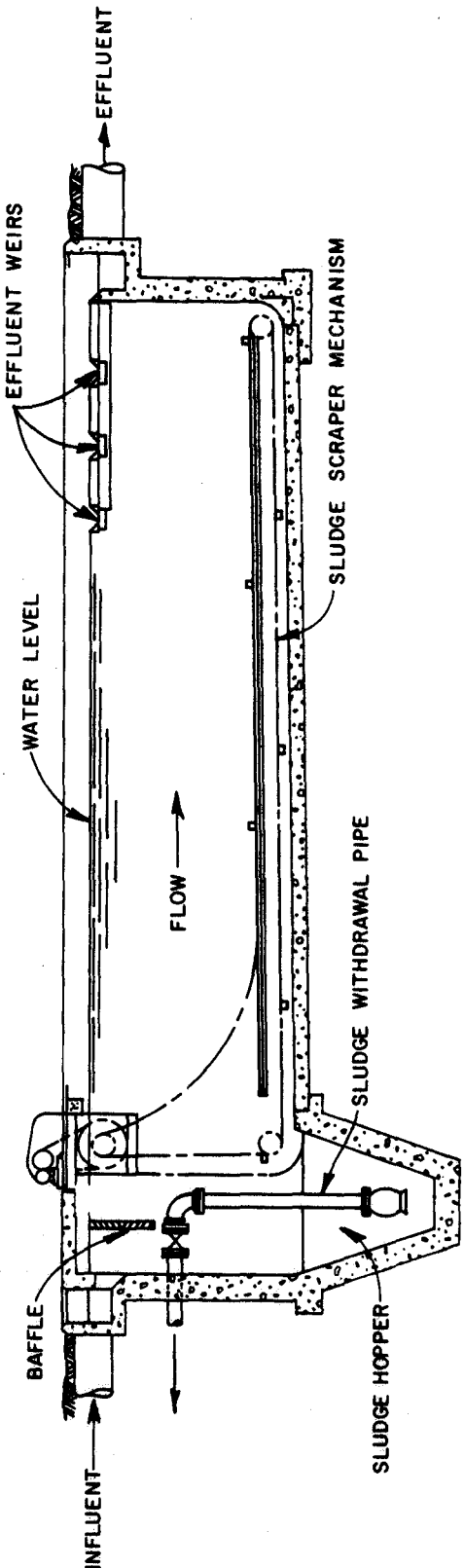

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FIGURE 3-1. RECTANGULAR SEDIMENTATION BASIN

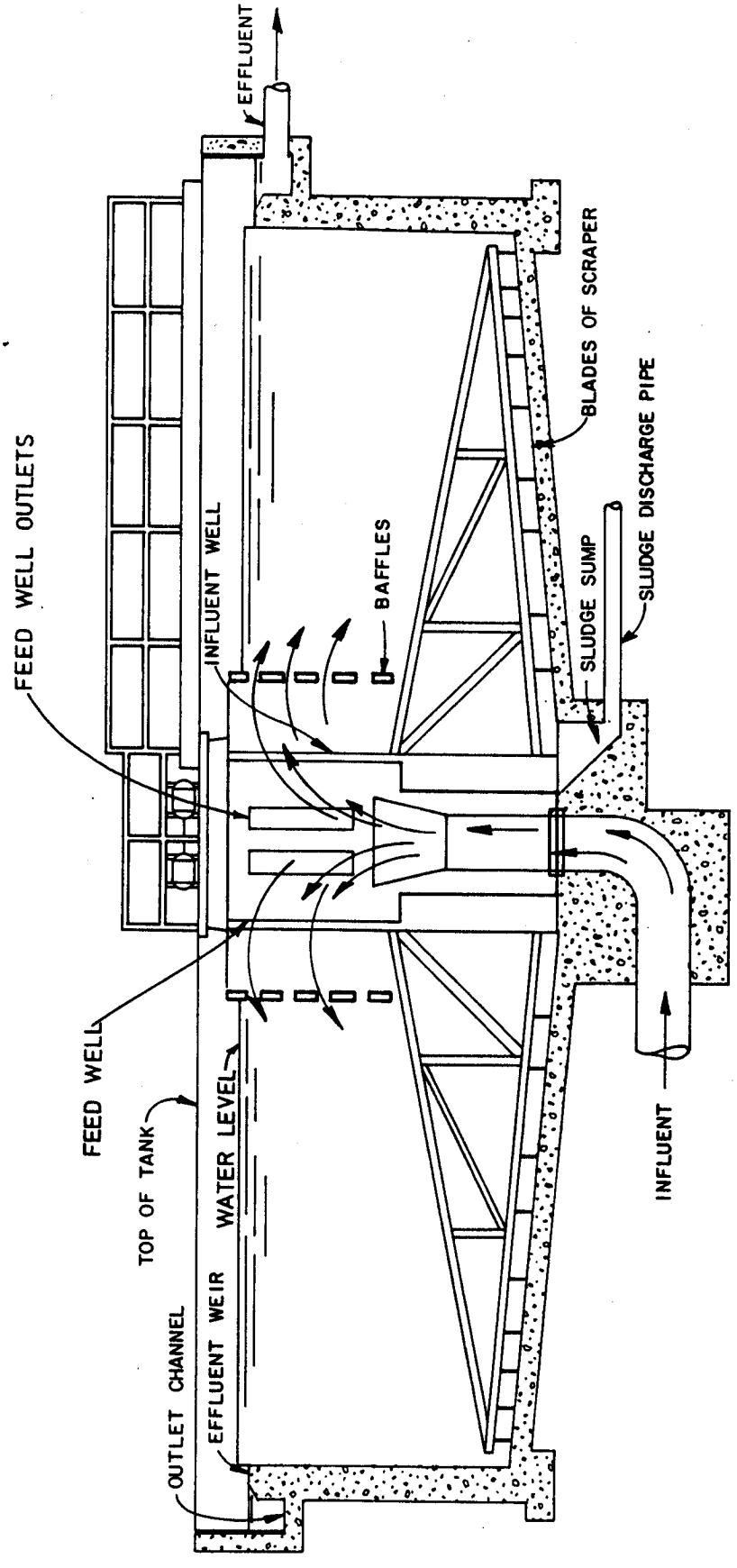

FIGURE 3-2. CIRCULAR SEDIMENTATION BASIN

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design must include arrangements that permit use of a single basin when necessary.

a. Design criteria. Basins may be rectangular, square, or circular in plan. Basin depths generally are from 10 to 16 feet. Under comparable conditions, deeper basins usually perform better than shallow ones. In general, the basins should be sized to provide a minimum detention time of 2 to 4 hours. For conventional coagulation plants, basin overflow rates commonly range from roughly 500 to 1,500 gpd per square foot with weir loading rates at 20,000 gpd per foot or less. Higher overflow rates may be used at some plants employing upflow clarification units. Sedimentation basin inlets, outlets, and baffles must be carefully designed to minimize short circuiting. The solids (sludge) which settle to the bottom of the basin must be removed before the depth of the sludge becomes great enough to interfere with effective sedimentation. In general, mechanical sludge removal equipment is a requirement. All basins must be provided with drains and overflows. Basin covers, usually in the form of a superstructure, will be required in northern localities where ice is a problem.

b. Flocculation-sedimentation basins. Units of this type, usually circular, combine the functions of flocculation, sedimentation, and sludge removal. Flocculation is accomplished in a circular center well. Sedimentation occurs in the annular space between the flocculation section and the perimeter effluent weir. Design criteria are generally similar to those applicable to separate units.

c. Suspended solids contact basins. Basins of this type combine rapid-mixing, flocculation, sedimentation, and sludge removal in a single unit. Coagulation and flocculation take place in the presence of a slurry of previously formed precipitates which are cycled back to the mixing and reaction zone. Upflow rates at the point of slurry separation should not exceed about 1.0 gpm per square foot for units used as clarifiers following coagulation.

3-6. Filtration. Filtration of water removes colloidal and larger particles from water by passage through a porous medium, usually sand or granular coal. The suspended particles removed during filtration range in diameter from about 0.001 to 50 micrometers and larger. Several different types of medium arrangements and rates of flow through filters can be used. The filtration process most commonly used is rapid-sand filtration, but pressure filters and diatomite filters are used at smaller installations.

a. Rapid sand filters.

(1) Filtration rate. Rapid sand filters commonly operate at rates between approximately 2 and 8 gpm per square foot. The usual rate is about 3 to 5 gpm per square foot. Adequate pretreatment and filter design will allow application rates of up to 6 gpm per square

foot with little difference in water quality. If high rates are to be used in design, great care must be taken to insure that all prefiltration treatment processes including coagulation, flocculation, and sedimentation will perform satisfactorily and consistently. High-rate filter operation definitely requires excellence in pre-filtration treatment, especially in the case of surface waters. It is recommended that data from laboratory or pilot studies be utilized whenever possible rather than an arbitrary selection of criteria.

(2) Filter medium.

(a) Sand. Silica sand is the most commonly used filter medium and its depth should be at least 24 inches and not more than 30 inches. When sand is employed, its "effective size" should fall in the range of 0.35 to 0.5 millimeter with a uniformity coefficient of 1.3 to 1.7. Filter sand usually ranges in size from that passing a 16-mesh (U.S. Series) sieve to that retained on a 50-mesh (U.S. Series) sieve. Approximately 100 percent by weight of the sand should pass the 16-mesh sieve and 90 to 100 percent be retained on a 50-mesh sieve. Filter sand should be clean silica sand having a specific gravity of not less than 2.5. The hydrochloric acid solubility of the sand should be less than 5 percent.

(b) Anthracite. Anthracite is an alternative medium consisting of hard anthracite coal particles. Its depth is usually between 8 inches and 3 feet. The effective size commonly ranges from about 0.45 millimeter to 0.6 millimeter with a uniformity coefficient not to exceed 1.7. The hardness should not be less than 2.7 on the Moh scale and the specific gravity not below 1.4. Also, the anthracite should be visibly free of clay, shale, and dirt.

(c) Multimedia. Multimedia filters employ two or three layers of media of different size and specific gravity. A common arrangement, the dual media filter, is 20 inches of anthracite overlaying a sand layer of approximately 8 to 12 inches. The anthracite layer has a size range of about 0.8 to 2.0 millimeter; the sand layer, about 0.4 to 1.0 millimeter. Tri-media filters employ an 18-inch anthracite layer, an 8-inch sand layer, and an underlying 4-inch layer of garnet or ilmenite having a size range of 0.2 to 0.4 millimeter. Garnet has a specific gravity of about 4 and ilmenite about 4.5.

(3) Filter gravel and underdrains. The filter media is commonly supported by a 10 to 18-inch layer of coarse sand and graded gravel. The gravel depth may range from 6 to 24 inches, depending on the filter underdrain system chosen. The gravel should consist of hard, rounded stones having a specific gravity of at least 2.5 and a hydrochloric acid solubility of less than 5 percent. A 3 to 4-inch transition layer of coarse (torpedo) sand, having a size range of about .05 to 0.1 millimeter, is placed on top of the filter gravel. Gravel size usually

ranges from about 0.1 inch to about 2.5 inches; fine material then increasingly coarser material in the direction of flow for filtration. Filter underdrains may be constructed of perforated pipe grids or various commercial underdrain systems.

(4) Number of filters. Not less than two filters should be installed regardless of plant size. For large plants, rough guidance as to the number of filters to be provided may be obtained from the following equation:

$$N = 2.7\ (Q)^{1/2}$$

where:

N = number of filter units
Q = design capacity in mgd

Thus, a 9 mgd plant would require eight filters.

(5) Size of filter units. The maximum filter size is related to wash water flow rate and distribution. Normally, individual filter sizes do not exceed about 2,100 square feet corresponding to a capacity of about 6 mgd at a flow rate of 2.0 gpm per square foot. A unit of this size would require a maximum backwash water rate of about 60 mgd, which is excessive. Consequently, it should be divided into two parts of equal size arranged for separate backwashing. Total filter box depth should be at least 9 feet.

(6) Filter backwash. Backwash facilities should be capable of expanding the filter media 50 percent. This will require wash rates in the range of 10 to 20 gpm per square foot. Backwash water can be supplied by a backwash pump or from elevated storage provided specifically for this purpose. Filter down-time during wash periods commonly average 10 to 20 minutes including a 5- to 15-minute wash period. For a 15-minute backwash of a single unit, at maximum rate, the wash water volume will be 300 gallons per square foot of filtration area in that unit. This aids in cleaning the filter and is commonly accomplished by rotary or fixed surface-wash equipment located near the top of the bed. It is operated for a time period equal to that of the backwash. Water pressures of 40 to 100 psi are required for surface-wash operation at a rate of 0.5 gpm per square foot. If an independent wash water storage tank is used, it must refill between washes. Tank capacity should be at least 1.5 times the volume required for a single wash.

(7) Wash water troughs. Wash water troughs equalize the flow of wash water and provide a conduit for removal of used water. Two or more troughs are usually provided. The elevation of the trough bottoms should be above that of the expanded bed. The clear horizontal distance between troughs should not exceed 5 to 6 feet, and the top of

the troughs not more than 30 inches above the top of the non-expanded bed.

(8) Filter piping and equipment. Essential filter control valves, etc., are shown schematically in figure 3-3. Each filter should be equipped with a rate-of-flow controller plus associated equipment for automatic filter water-level control. The latter senses the water level in the main influent conduit and transmits a signal to the flow controllers. The controllers, in response to this signal, adjust filtration rates to match the inflow from the sedimentation basins. Thus, within practical limits, total filter outflow always equals total inflow and the filter water level remains virtually constant. A device that will sense maximum permissible clearwell level should also be provided. This should be arranged so that at maximum allowable clearwell water level a shut-off signal will be transmitted to all filter controllers and also to an audible alarm. Other designs, not involving rate controllers, such as "influent flow splitting" and "variable declining rate" have been developed and may be employed at the discretion of the designer. In general, each filter must have five operating valves: influent, wash water, drain, filtered water, and filter-to-waste. It is emphasized that the filter-to-waste piping must not be directly connected to a plant drain or sewer. An effluent sampling tap must be provided for each filter. Valves can be manually, electrically, hydraulically, or pneumatically operated. Butterfly type valves are recommended for filter service. Design velocities commonly employed for major filter conduits are as follows:

Conduit	Design Velocity fps
Influent	1-4
Filtered water	3-6
Wash water	5-10
Drain	3-8
Filter-to-waste	6-12

The filtered water conduit must be trapped to prevent backflow of air and provide a seal for the rate controllers. The filter pipe gallery should have ample room, good drainage, ventilation, and lighting. Dehumidification equipment for the gallery should receive consideration. Filters should be covered by a superstructure except under favorable climatic conditions. Drainage from the operating floor into the filter should be prevented by a curb. Access to the entire bed should be provided by a walkway at operating floor level around the filter. Filters may be manually or automatically controlled from local or remote locations. Facilities permitting local, manual control are recommended irrespective of other control features. Used backwash water should be discharged to a wash water recovery basin or to a waste disposal facility. Regulatory agencies generally view filter wash

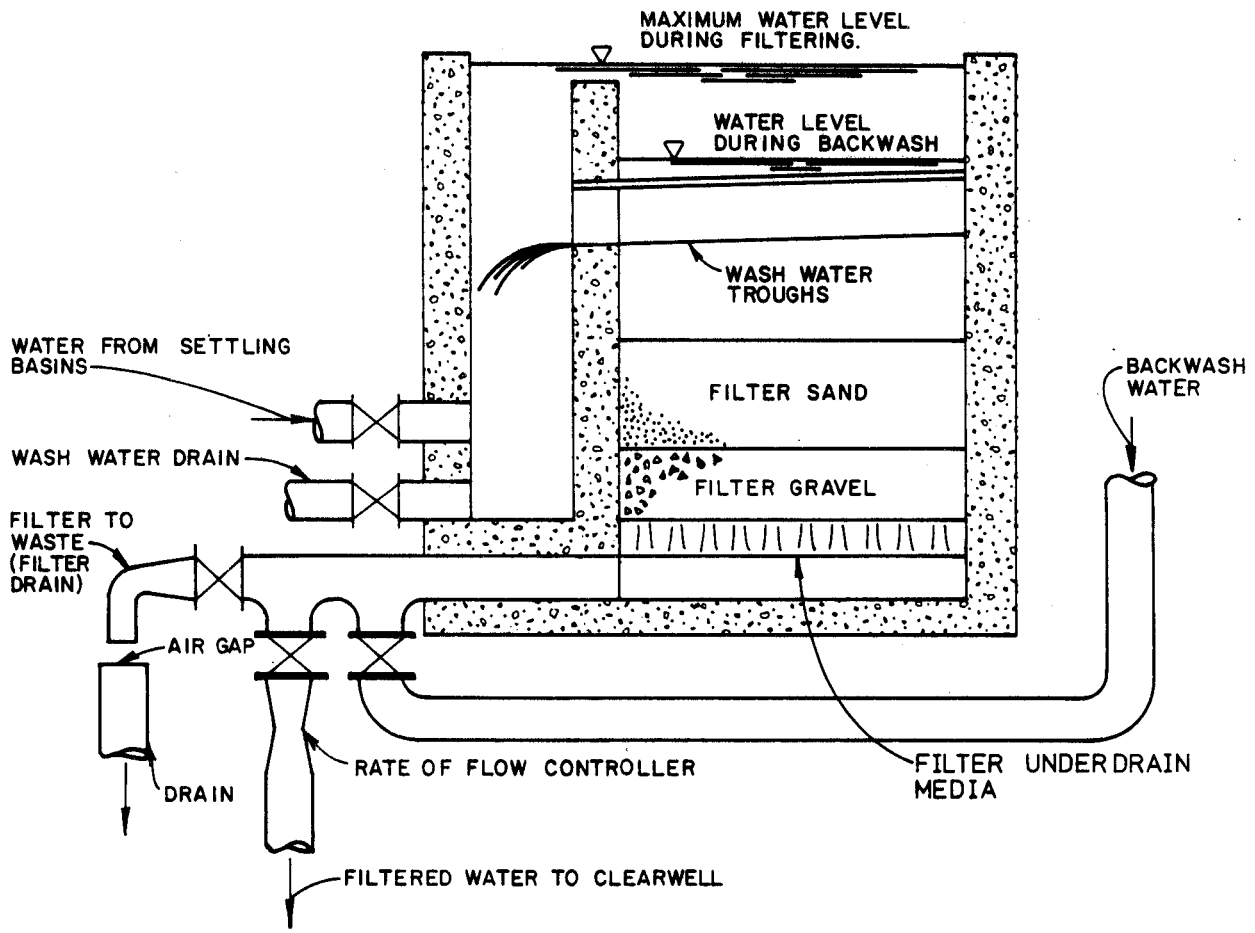

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FIGURE 3-3. SCHEMATIC OF RAPID SAND FILTER OPERATIONAL CONTROLS

water as a pollutant and forbid its direct discharge to the natural drainage.

(9) Essential instrumentation. Minimum essential instrumentation for each filter will be provided as follows: rate-of-flow indicator, loss-of-head indicator, wash water rate-of-flow indicator, and totalizing meter. If a wash water storage tank is provided, it must be equipped with a water-level indicator.

b. Pressure filters. Pressure filters are similar in construction and operating characteristics to rapid sand filters; however, in a pressure filter the media, gravel bed, and underdrains are enclosed in a steel shell. An advantage of a pressure filter is that any pressure in the waterlines leading to the filter is not lost, as in the case of gravity filters, but can be used for distribution of the filtered water. Between 3 and 10 feet of pressure head are lost through the filter. The primary disadvantage of a pressure filter is that, due to the filter being enclosed in a steel shell, access to the filter bed for normal observation and maintenance is restricted. Also, the steel shells require careful periodic maintenance to prevent both internal and external corrosion. The use of pressure filters is not advantageous in most systems; however, if the pressure requirements and conditions in a particular system are such that repumping of filtered water can be eliminated, cost savings will be realized.

3-7. Disinfection. Disinfection involves destruction or deactivation of organisms which may adversely affect the health of the water consumer. The preferred method of disinfecting water supplies is by application of chlorine to water (chlorination).

a. Definitions. Terms frequently used in connection with chlorination practices are defined as follows:

(1) Chlorine demand. The difference between the concentration of chlorine added to the water and the concentration of chlorine remaining at the end of a specified contact period. Chlorine demand varies with the concentration of chlorine applied, time of contact, temperature, and water quality.

(2) Chlorine residual. The total concentration of chlorine remaining in the water at the end of a specified contact period.

(3) Combined available residual chlorine. Any chlorine in water which has combined with nitrogen.

(4) Free available residual chlorine. That part of the chlorine residual which has not combined with nitrogen.

b. Chlorination practice.

(1) Breakpoint chlorination. If water contains ammonia or certain nitrogeneous organic matter which reacts with chlorine, the addition of chlorine causes the formation of chloramines until the ratio of elemental chlorine to ammonia compounds is about 5 to 1. Further addition of chlorine results in the oxidation of chloramines to gaseous nitrogen and nitrogen oxides, which decreases the quantity of chloramines present. After all of the chloramines have been oxidized, additional chlorine added to the water forms only free available chlorine. The point at which all of the chloramines have been oxidized and only free chlorine is formed is called the "breakpoint." If no ammonia is present in the water, there will be no breakpoint. The chlorine required to reach the breakpoint is usually about 10 times the ammonia nitrogen content of the water. However, in certain waters, because of the presence of other chlorine consuming substances, as much as 25 times the ammonia nitrogen concentration may be required. Enough chlorine should be added past the breakpoint to insure an adequate free chlorine residual.

(2) Marginal chlorination. Marginal chlorination involves the application of chlorine to produce a desired level of total chlorine residual regardless of the relative concentrations of free or combined chlorine present. In marginal chlorination, the initial chlorine demand has been satisfied, but some oxidizable substances remain.

(3) Chlorine dosages. Figure 3-4 provides minimum cysticidal and bactericidal free chlorine residuals and minimum bactericidal combined chlorine residuals for various pH and temperature levels. Since waterborne bacteria are the major concern at fixed installations, minimum bactericidal levels will be maintained in treated water in all parts of the distribution system under constant circulation. Even at lower pH levels, free chlorine residuals should not fall below 0.2 mg/l and combined chlorine residuals should not fall below 2.0 mg/l. If marginal chlorination is practiced, the total chlorine residual must not be less than 2.0 mg/l. Whenever epidemological evidence indicates an outbreak of a nonbacterial waterborne disease such as amebiasis, infectious hepatitis, or schistosomiasis, cysticidal free chlorine residuals must be maintained in the water supply.

c. Other effects of chlorination. In addition to the disinfection achieved with chlorination, many other effects should be noted. Since the oxidizing power of chlorine is high, in the presence of free chlorine, hydrogen-sulfide is oxidized, nitrites are oxidized to nitrates, and soluble iron and manganese are oxidized to their insoluble states. Free chlorine also destroys many taste and odor producing substances. Many organic substances will react with chlorine to form chloro-organic compounds which may be objectionable either because of resultant tastes and odors or because of possible health effects.

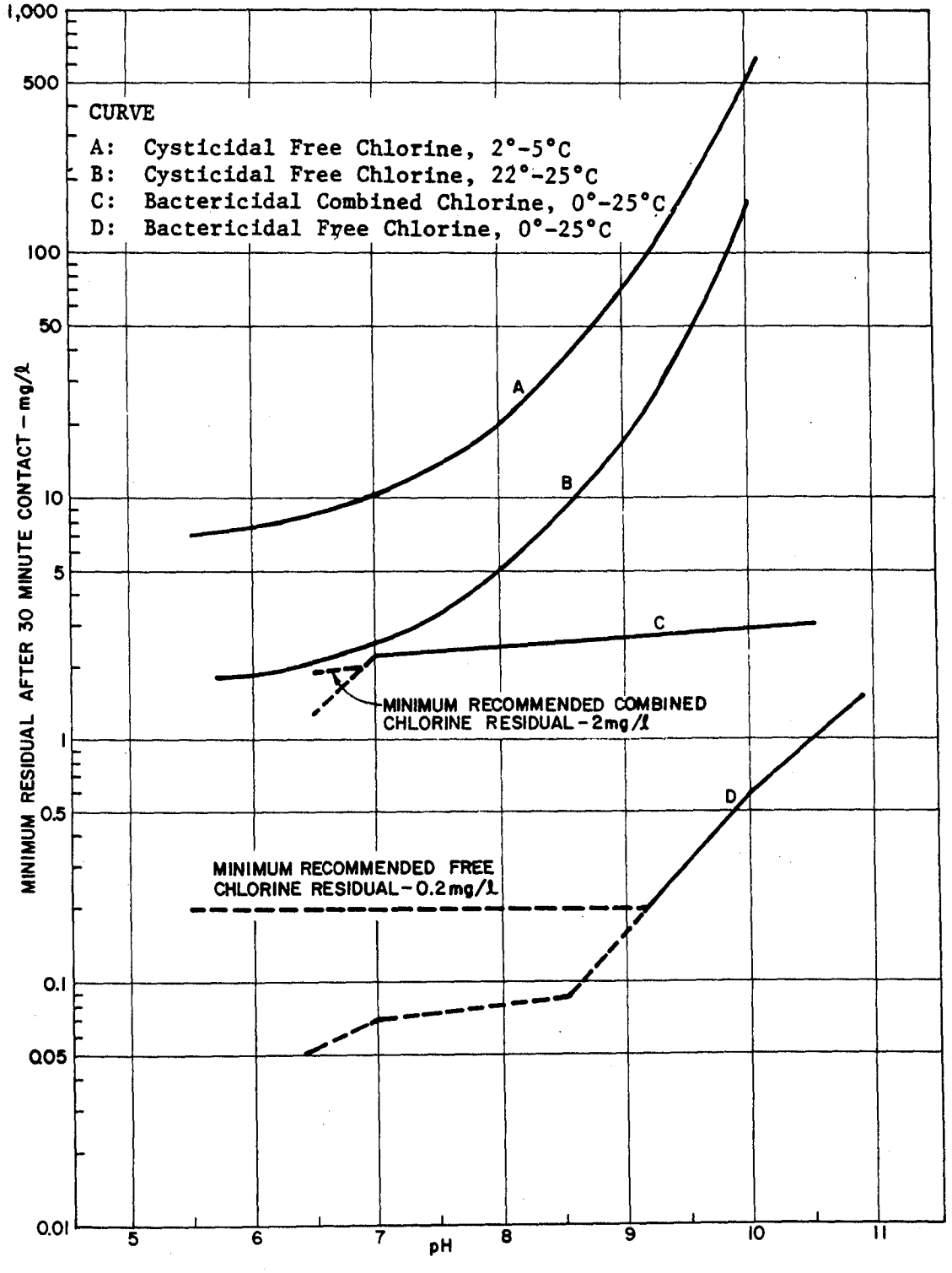

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FIGURE 3-4. MINIMUM CHLORINE RESIDUALS

d. Application of chlorine. Chlorine may be applied to water in one of two forms: as elemental chlorine or as hypochlorite salts. Hypochlorite salts should be used for water disinfection at all fixed installations unless it can be demonstrated that the use of elemental chlorine is more advantageous.

(1) Point of application. Chlorine may be applied to water in a variety of locations in the water treatment plant, storage facilities, or distribution system. It is absolutely essential that the chlorine applied to the water be quickly and thoroughly mixed with the water undergoing treatment. If required, special chlorine mixing facilities should be provided. In conventional water treatment plants, chlorine may be applied prior to any other treatment process (prechlorination), following one or more of the unit treatment processes (postchlorination), or in the more distant points of the distribution system (rechlorination).

(2) Chlorination equipment. Hypochlorite salts must be applied to the water in solution form. Hypochlorite solutions are pumped by a diaphragm pump through an injection system into the water to be chlorinated. If elemental chlorine is used for disinfection, it should be injected by solution-type chlorinators. Since chlorine solutions are corrosive, many components of a chlorination system must be constructed of corrosion resistant materials such as glass, silver, rubber, or plastics. Maintaining the chlorination apparatus in a trouble-free state is essential; therefore, key spare parts and repair kits for chlorination systems will be kept on hand.

(3) Automatic control. If automatic chlorination control is utilized, the chlorine feed rate should be controlled primarily by the rate of flow of water, with a signal from a downstream residual chlorine analyzer used to adjust the feed rate. Provision for manual control during emergency situations must be included.

e. Superchlorination and dechlorination. Superchlorination may be necessary if there are large variations in chlorine demand or if available contact time is brief. Water which has been superchlorinated generally requires dechlorination before delivery to the distribution system. Dechlorination may be achieved through the application of sulfur dioxide, sodium bisulfite, or sodium sulfite, or by passing the water through granular activated carbon filters. The dechlorination process (and subsequent rechlorination, if necessary) should be controlled so that the free residual chlorine remaining in the water is at least 0.2 mg/l.

f. Safety precautions for chlorination. The AWWA M3 contains safety recommendations regarding the use of chlorine. These recommendations will be followed at all Army water treatment facilities.

3-8. Taste and odor control. Most taste and odors in surface water are caused by low concentrations of organic substances derived from decomposing vegetation, microscopic organisms, sewage and industrial waste pollution, etc. Treatment for taste and odor removal involves destruction of the odorous substance by chemical oxidation or its removal by aeration or adsorption by activated carbon.

a. Chemical oxidation. Chemical oxidizing agents which have been found effective and which can be used in the treatment of potable water are chlorine, chlorine dioxide, potassium permanganate, and ozone. No single chemical is completely effective under all operating conditions.

b. Aeration. Aeration is helpful in eliminating odor caused by hydrogen sulfide and other dissolved gases, but is ineffective in significantly reducing odor associated with dissolved organics.

c. Adsorption. Powdered activated carbon is commonly used for removal of odor by adsorption. The carbon can be applied to the water at any point in the treatment plant prior to filtration, but it is usually advisable to apply it early in the treatment process. Sometimes a split feed of carbon is employed with a portion applied to the raw water and a smaller amount to the filter influent. For maximum effectiveness, carbon should be applied well ahead of chlorination. The influent to a presedimentation basin is normally an effective carbon application point. Powdered carbon dosages usually range from 5 to 10 mg/l, but as much as 50 mg/l may be required under adverse conditions. Granular carbon has also been used for taste and odor removal. It has been employed as a separate treatment step in the form of carbon columns and as a substitute for sand in the filtration process. Used in this way, the granular carbon serves in a dual capacity as a filtration medium and for taste and odor removal.

3-9. Softening.

a. Applications. Softening of the entire base water supply will be considered only if the hardness exceeds 400 mg/l, expressed as equivalent $CaCO_3$. In this case, conventional softening techniques, which can be found in standard textbooks on water treatment processes should be considered. Softening to levels below 400 mg/l will only be performed on waters intended for the applications outlined in paragraphs (1) through (3) below. The cation ion exchange or zeolite process should be considered here unless other treatment methods are more feasible or necessary.

(1) Laundries. Water for laundries should have a hardness of 50 mg/l or less.

(2) Boilers. Boiler water for power plants and heating plants may require softening, but satisfactory results can often be obtained

by application of corrosion and scale inhibitors such as sodium sulfite, sodium hexametaphosphate, and trisodium phosphate. Depending on the pressure at which the boiler is to operate, partial water-demineralization may also be necessary. See paragraph 3-12a for additional information on demineralization.

(3) Hospitals. When the water supplied to a hospital has a hardness of 170 mg/l or more, the water will be softened to approximately 50 mg/l. Where critical equipment requires water having a hardness of less than 50 mg/l, the most feasible means of obtaining that water quality will be used. Zero hardness water may be piped from the main softener or may be supplied from small individual softeners, whichever is the more feasible. The sodium content of the treated water must be taken into account when selecting a softening method for hospitals.

b. Cation exchange softening. Hardness is caused principally by the cations of calcium and magnesium, and cation exchange softening is accomplished by exchanging these ions for a cation, usually sodium, which does not contribute to hardness. This exchange is achieved by passage of the water through the bed of a granular sodium cation exchanger. This reaction is reversible and the exchanger can be regenerated with a strong solution of sodium chloride (common salt).

(1) Media used for ion exchange softening. Ion exchange water softeners should use polystyrene resins as the softening media. Such resins must have a hardness exchange capacity of at least 25,000 grains of hardness per cubic foot of resin.

(2) Regeneration of ion exchange softeners. The regeneration process generally involves three steps: (1) backwashing, (2) application of regenerating solutions, and (3) rinsing.

(a) Backwashing. The purposes of water softener backwashing are generally the same as the purposes of filter backwashing. Any turbidity particles filtered out of the water during softening are removed by the backwashing process. For polystyrene resin media, bed expansions of from 50 to 100 percent are normally required, which involve backflow rates of 4 to 10 gpm per square foot of bed area. Backwash periods generally range from 2 to 5 minutes. Ion exchange water softeners which operate upflow rather than downflow will not require backwashing, but the water to be softened must be virtually free of suspended matter.

(b) Application of salt brine. After the unit has been backwashed, a salt solution is applied to the medium in order to regenerate its softening capabilities. Regeneration brines should be 10 to 15 percent solutions of salt. The more salt used in the regeneration of a softener, the more complete the regeneration will be, and the greater the exchange capacity of the regenerated medium will

be. Salt consumption commonly ranges from about 0.3- to 0.5-pound of salt per 1,000 grains of hardness removed. The contact time of the brine with the softening medium also has a direct effect on the exchange capacity of the regenerated medium. Contact times of 20 to 35 minutes will generally be used.

(c) Rinsing. After regeneration, the brine must be rinsed from the unit before softening is resumed. Backwash water, spent regenerant, and rinse water must be disposed of properly.

(3) Ion exchange water softeners. Although most ion exchange softeners at Army installations will be downflow pressure softeners, softening can also be achieved upflow. Larger ion exchange softening facilities are often operated upflow in order to avoid the necessity of backwashing. In general, ion exchange softeners are of two types: open gravity softeners and pressure softeners.

(a) Open gravity softeners. Open gravity softeners are constructed in much the same manner as rapid sand filters, and the modes of operation are very similar. However, the ion exchange medium used in open gravity softeners is much lighter than the sand used in filters, so backwash rates for open gravity softeners may also be operated upflow, but the softener will not achieve any filtering effects so the influent water must be virtually free of suspended matter.

(b) Pressure softeners. A polystyrene resin medium used for pressure softening should have a minimum bed depth of 24 inches and physical properties approximately the same as the following:

Shipping weight	45-55 pcf, net
Density	48-53 pcf
Moisture content	42-48 percent
Void volume	40 percent
Screen grading	16-50 mesh
Effective size	0.45-0.55 millimeter
Uniformity coefficient	<1.70
Fines through 50 mesh screen	<1 percent

If downflow softening is used, the flow rate through the softening medium may vary from 2 to 8 gpm per square foot but must not exceed 10 gpm per square foot under the most severe loadings. Severe reductions in exchange capacity are experienced if the softener operates at rates of flow in excess of 10 gpm per square foot for sustained periods of time. With upflow softening, the rate of flow should be adjusted to maintain a bed expansion of from 40 to 60 percent. The degree of bed expansion is a function of both the flow rate and the temperature of the influent water, so the flow rate must be decreased as water temperature decreases if a constant bed expansion is to be maintained.

(4) Blending. An ion exchange softener operating properly will produce a water having a hardness approaching zero. Inasmuch as it is not generally economical nor desirable to soften all water to this low hardness level, provisions for blending the softened water with the unsoftened water are desirable.

(5) Other factors affecting ion exchange softening.

(a) Turbidity. Turbidity particles present in the water influent to the softener are deposited on the softening medium and may cause losses of exchange capacity and excessive head losses through the softener. If turbidity levels are excessive, the particles must be removed from the water prior to softening or special backwashing procedures must be implemented.

(b) Bacterial slimes. Unless proper disinfection is practiced, bacterial slimes can form in the softening medium and cause excessive head losses and loss of exchange capacity. These slimes can be prevented or removed through chlorination of feedwater or regeneration water.

(c) Temperature. The loss of head through a water softener is strongly affected by water temperature, with lower head losses occurring at higher temperatures. For example, at similar flow rates, the head loss through a softener at 122 degrees F. is only about 35 percent of what it would be at 36 degrees F. Also, the water temperature affects the exchange capacity of the softener, with a 10 to 15 percent increase at high operating temperatures (>86 degrees F.) over the exchange capacity at low temperatures (32 to 50 degrees F.).

(d) Iron, manganese, and aluminum. If iron, manganese, and aluminum are present in the influent water, precipitates may be formed which coat the ion exchange media particles and cause a loss of exchange capacity. This problem can be avoided through treatment to remove the iron, manganese, and aluminum from the water prior to softening.

(e) Total hardness and sodium concentration. If the total hardness exceeds 400 mg/l or the sodium salts, expressed as mg/l $CaCO_3$, exceed 100 mg/l, the softener should be sized on the basis of the "compensated total hardness" rather than the total hardness. Compensated hardness is calculated from the equation:

$$TH_C = \frac{(TH)\ (9{,}000)}{9{,}000 - TC}$$

where:

TH_c = compensated hardness in mg/l as $CaCO_3$
TH = total hardness in mg/l as $CaCO_3$
TC = total cations in mg/l, all expressed as $CaCO_3$

Compensated hardness (TH_c) in mg/l is converted to grains per gallon by multiplying by 0.0584 or dividing by 17.1.

3-10. Iron and manganese control.

a. Occurrence of iron and manganese. Dissolved iron and manganese are encountered principally in ground waters devoid of dissolved oxygen. Normally, oxygenated surface waters do not contain significant concentrations of these metals; however, stagnant water, found in the bottom of thermally-stratified reservoirs, sometimes contains dissolved iron and manganese. Their presence in solution is associated with anaerobic conditions near the bottom of the reservoir.

b. Effects of iron and manganese. Dissolved iron in excess of 1 or 2 mg/l will cause an unpleasant taste, and on standing, the water will develop a cloudy appearance. Iron concentrations appreciably greater than 0.3 mg/l will cause red stains on plumbing fixtures and laundry. Similarly, manganese will cause black stains if present to the extent of more than about 0.05 mg/l. Deposits of iron and manganese can build up in water distribution systems and periodic "flush-outs" of these deposits result in objectionable color and turbidity at the consumer's tap.

c. Removal by oxidation and filtration. Oxidation can be accomplished with dissolved oxygen, added by aeration, and by the addition of an oxidizing chemical, such as chlorine, chlorine dioxide, potassium permanganate, or ozone. Manganese is more difficult than iron to oxidize and precipitate. In the absence of manganese, iron can often be removed with minimum treatment, consisting of aeration followed by direct filtration. In general, aeration alone will not oxidize manganese unless the pH is raised to about 9.5. Strong oxidants, such as chlorine or potassium permanganate, are effective at lower pH values. To insure oxidation, precipitation, and agglomeration of iron and manganese and their essentially complete removal, at least three treatment steps are usually necessary: aeration, contact time, and filtration. An aerator containing trays of coke, limestone, etc., is commonly used. Reaction time is provided by a contact or contact-sedimentation basin having a detention period of at least 30 minutes. Filtration is accomplished by conventional single or multi-media filters designed for a filtration rate of at least 3.0 gpm per square foot. The aeration step is frequently supplemented by a chemical oxidant, such as chlorine or permanganate. Flocculation is advantageous in the contact basin, particularly if iron exceeds about 2 mg/l.

3-11. Corrosion and scale control. "Corrosion" can be defined as the deterioration of metal by direct chemical or electrochemical reaction with its environment. "Scale" refers to an accumulation of solids precipitated out of the water. In water treatment, corrosion and scale are closely associated. Both must be considered in connection with the design and operation of the treatment works. Thin scale may be desirable because it can provide a measure of protection against corrosion. However, thick layers of scale are detrimental in both hot and cold water systems. It is desirable to produce a "balanced" water that is neither highly corrosive nor excessively scale forming.

a. Corrosion.

(1) The extent and nature of corrosion reactions depend upon many factors. Among the most important are the chemical and physical nature of the water, its velocity, pipe metallurgy, and pipe coating. Controllable factors are principally calcium content, alkalinity, and pH. Certain corrosion inhibitors can also be used, but relatively few are suitable for potable water systems.

(2) Treatment to insure deposition and maintenance of a thin layer of calcium carbonate on the pipe interior is one widely used means of corrosion control. The rate of formation of calcium carbonate is favored by high concentrations of calcium and bicarbonate and carbonate alkalinity. Protection of this type cannot be attained in waters containing very low concentrations of calcium and alkalinity.

(3) Corrosion rates may also be reduced by the use of certain inhibitors. For potable water systems, the most practicable inhibitors are silicates and certain polyphosphate compounds.

(4) Dissolved oxygen and carbon dioxide have a significant effect on corrosion rates. Carbon dioxide lowers the pH and makes the water more aggressive.

(5) Corrosion rates are influenced to some extent by all mineral substances found in water, but corrosion effects are so interrelated that it is not possible to isolate the quantitative influence of individual ions. It is known that high concentrations of chloride and sulfate ions will produce increased corrosion rates. However, their adverse effects are somewhat mitigated by alkalinity (carbonate, bicarbonate) and calcium ions. Excessive calcium and alkalinity will often result in objectionable scale formation. It is, therefore, necessary to seek a compromise between corrosion on the one hand and scale formation on the other.

(6) Based on the corrosion accelerating effects of chloride and sulfate and the corrosion inhibiting effects of alkalinity, the following ratio, termed the "Corrosion Index," has been developed.

$$\text{Corrosion Index} = \frac{[.02821]\ [Cl^-] + [.02083]\ [SO_4^{2-}]}{[.01639]\ [HCO_3^-]}$$

where:

$[Cl^-]$ = mg/l chloride ion as Cl^-

$[SO_4^{2-}]$ = mg/l sulfate ion as SO_4^{2-}

$[HCO_3^-]$ = mg/l bicarbonate ion as HCO_3^-

For a pH range of about 7 to 8, and in the presence of dissolved oxygen, an index below about 0.1 indicates probable general freedom from corrosion. An index higher than 0.1 is indicative of corrosive tendencies. The higher the index, the greater the probability of corrosion.

b. Scale. Scale problems in distribution systems are caused principally by calcium carbonate, magnesium hydroxide, aluminum hydroxide, and the oxides and hydroxides of manganese and iron.

(1) Aluminum. Aluminum hydroxide deposits can result from excessive alum use for coagulation and/or improper coagulation practice, such as poor mixing and flocculation and incorrect coagulation pH.

(2) Magnesium. Magnesium hydroxide deposits have caused serious difficulties in distribution systems and hot water heaters. Magnesium solubility is highly sensitive to pH and temperature and failure to exercise careful control over its stabilization following softening will usually lead to deposition problems. It is advisable to maintain magnesium hardness below 40 mg/l and pH below 9.8. Hot water heaters should be operated so that water temperatures will not exceed 140 degrees F.

c. Chemical control of scale and corrosion. Corrosion protection of water system components can be achieved through the use of corrosion-resistant materials including plastics, copper, concrete, and cement-asbestos; the use of galvanized pipe; and the use of cement or coal-tar or calcite lined pipe. Chemical control by pH adjustment, and/or the addition of sodium silicate or polyphosphates to the water may be employed to supplement these protective measures if it is probable scale and/or corrosion will pose major problems to the water system components over the project design life. Chemical control can be expected to alleviate the problems but not necessarily eliminate them.

3-12. Special processes. In some cases, it will be necessary to use raw water supplies containing unacceptably large concentrations of constituents that cannot be removed by conventional treatment

processes. The most common of these objectionable constituents are mineral salts, such as sulfates and chlorides, dissolved organic substances, and excess fluorides. Special treatment processes are necessary to remove these materials.

a. Demineralization. The presence of excessively high concentrations of dissolved materials in water is indicated by high chloride (Cl^-), sulfate (SO_4^{2-}), and total dissolved solids (TDS) levels. The limits for these substances for mobilization work are 250 mg/l, 250 mg/l, and 500 mg/l, respectively. These limits are based on esthetic considerations, and considerably higher concentrations, while not desirable, can be tolerated. Where demineralization is required, processes commonly employed are electrodialysis, reverse osmosis, distillation, and ion exchange. Waste brine solutions derived from these processes must be disposed of properly. All demineralization processes are energy intensive, and alternative water sources should be thoroughly investigated before a commitment to a demineralization project is made. If the demineralization process selected requires large inputs of electricity, consideration should be given to its operation principally during "off-peak" hours with storage of desalted water until needed.

b. Removal of dissolved organics. An effective means of removing dissolved organics is by their adsorption on granular activated carbon. Periodically, the granular carbon must be removed from the treatment system and reactivated in a special furnace. The carbon can serve as a filter medium as well as an adsorbent and is sometimes installed in conventional filters in place of the sand filter medium. Alternatively, the granular carbon may be employed in a special carbon contactor following conventional filtration. The granular carbon's effective size should be 0.7 to 1.0 millimeter; its uniformity coefficient, <1.9. When used in a filter, the carbon bed depth should be at least 2 feet and the corresponding hydraulic loading about 2 gpm per square foot. Where specially designed carbon contactors are provided following conventional filtration, 2- to 10-foot bed depths and hydraulic rates of about 5 gpm per square foot can be employed. The longer contact time provided by the deeper beds reduces reactivation frequency.

c. Excess fluoride removal. Fluoride concentration in excess of the limits stipulated in TB MED 576 pose a health hazard and must be removed. Fluoride removal can be accomplished by passage of the water through beds of activated alumina, bone char, or tricalcium phosphate, electrodialysis, reverse osmosis, or ion exchange. All removal processes produce liquid wastes and suitable provisions must be made for their disposal. Guidance as to the fluoride removal process to be employed can be obtained from laboratory studies of process effectiveness and fluoride removal capacity, using samples of the water that is to be treated.

CHAPTER 4

MEASUREMENT AND CONTROL

4-1. Measurement of process variables. In order to determine the degree of effectiveness of the different treatment processes, several physical and chemical parameters associated with water treatment must be measured. After they are measured, the information must be evaluated so that necessary adjustments can be made in the treatment process.

a. Minimum analyses. The minimum type, number, and frequency of analyses for water treatment plants will conform to paragraph B-1 of appendix B.

b. Laboratories. Provisions should be made for laboratory facilities either on or off the mobilization site. If feasible, the services of private laboratories or laboratories at nearby municipal water plants should be solicited.

c. Records of analyses. Results of laboratory analyses will be recorded and maintained in an orderly filing arrangement.

4-2. Control. Water treatment plant processes may be controlled by manual, semiautomatic, or automatic methods, which are defined as follows.

a. Manual control. Manual control involves total operator control of the various water treatment processes. The personnel at the water treatment plant observe the values of the different variables associated with the treatment processes, and make suitable adjustments to the processes.

b. Semiautomatic control. Semiautomatic control utilizes instruments to automatically control a function or series of functions within control points that are set manually. The operator manually starts the automatic sequence of operations. An example of semiautomatic control is the automatic backwashing of a filter after operator initiation of the program.

c. Automatic control. Automatic control involves the use of instruments to control a process, with necessary changes in the process made automatically by the controlling mechanisms. When a process variable changes, the change is measured and transmitted to a control device which adjusts the mechanisms controlling the process. Automatic control systems have been developed which are reliable, but provisions for emergency manual control must be included.

4-3. Design of instruments and controls. All instruments and control devices should be placed in readily accessible locations in order to

facilitate observation, maintenance, repair, and replacement. Instruments should not be located in environments which might lead to premature failure of the instruments. Examples of such environments are areas subject to high temperatures or corrosive vapors. Provisions should be made for many of the instruments to actuate alarms if critical process variables exceed or fall below predetermined tolerable levels. Such alarms should include both audio and visual signals.

CHAPTER 5

WATER TREATMENT CHEMICALS

5-1. Chemical properties. Chemicals are used for a variety of purposes in conventional water treatment practice, including coagulation and flocculation, disinfection, taste and odor control, and pH adjustment. The most common chemicals and some of their characteristics are listed in table 5-1.

5-2. Chemical standards. Chemicals used at Army water treatment plants will meet the applicable standards of the American Water Works Association (AWWA). If chemicals meeting the AWWA standards are not available, then substitute materials may be used after they are evaluated and determined by the Surgeon General of the Army to be safe for drinking water.

5-3. Chemical handling and storage. In the design of water treatment facilities, the selection of methods of chemical handling and storage must be based primarily on ease of operation, operating flexibility, and safety considerations. If chemicals are to be received in shipping containers such as bags, boxes, drums, or canisters, equipment required for chemical handling may include carts, dollies, fork lifts, cranes, etc. If chemicals are shipped in bulk quantities, the mode of unloading depends on the physical characteristics of the chemical. Bulk liquids are usually unloaded by pumping from the tank truck or railroad car to the storage tanks at the treatment plant. Bulk powders can be unloaded by pneumatic unloading and conveyance devices, or if the powder is to be mixed or dissolved in water, it can be unloaded directly into a water eductor in which the powdered chemical and the water are mixed as the water is flowing to the storage tank. Chemical crystals or granules are usually unloaded by mechanical devices, such as bucket elevators and conveyor belts. All three forms of bulk chemicals can be unloaded by gravity if the chemical storage tanks or bins are located below ground near railroad tracks or roadway. Chemicals shipped in bags, drums, barrels, or other shipping containers can usually be stored by placing these containers in a specified storage area. The supply of chemicals in storage at a water treatment plant should always be at least equal to the projected 30-day requirements. Under some circumstances, it may be desirable to maintain larger supplies of essential chemicals, such as disinfectants or coagulants, and smaller supplies of nonessential chemicals. Extreme caution must be used when handling and storing most water treatment chemicals. Mishandling may cause death, injury, or at the very least, may render chemicals ineffective. Hazardous chemicals, such as chlorine gas, must be stored in separate rooms to avoid reaction of chemical vapors. When doubt exists as to the proper handling of certain chemicals, advice from the manufacturer or other technically competent authority should be sought.

5-4. Chemical application.

a. Dry chemicals. Dry chemicals are usually converted to a solution or slurry prior to application to the water. Measurement of the chemical application rate is accomplished by the dry-feed machine. The measured quantity of chemical is then dissolved or slurried in a small amount of water for transport to the feed point, where the solution or slurry must be rapidly and thoroughly mixed with water being treated.

b. Liquid chemicals. Chemical solutions or slurries are applied directly, or after dilution, to the water being treated by volumetric liquid feeders such as metering pumps or rotating wheel feeders. Rapid, thorough mixing of the chemical solution or slurry with the water is essential.

c. Chlorine application. Hypochlorite solution will be fed by an injector, diaphragm pump, or centrifugal pump system. The point of discharge of the chorine solution must be at least 2 feet below the water surface.

d. Corrosion. Special attention should be directed to the materials used for the critical parts of chemical feeders. Many chemicals form corrosive environments for common metals.

Table 5-1. Principal Chemicals Used in Water Treatment

Chemical Name and Formula	Common or Trade Name	Purpose	Shipping Containers	Suitable Handling Materials	Bulk Density lb/cu ft	Solubility lb/gal	Commercial Strength	Characteristics
Aluminum sulfate solution	Liquid alum	Coagulation	Tank trucks or tank cars	Lead or rubber-lined steel, 316 stainless steel, plastic	(11.047 lbs/gal)	-	5.36 lbs dry alum per gallon	pH of 1% solution is 3.4
Aluminum sulfate (dry), $Al_2(SO_4)_3 \cdot 14\ H_2O$	Alum, filter alum, sulfate of alumina	Coagulation	100-200 lb. bags, 300-400 lb. barrels, bulk (carloads).	Dry-Iron, steel. Solution-lead-lined rubber, silicon, asphalt, 316 stainless steel	Powder 38-45 Granule 60-63 Lump 62-67	4.2 at 60°F	15-22% Al_2O_3	pH of 1% solution is 3.4
Sodium aluminate $Na_2OAl_2O_3$	Soda alum	Coagulation	100-150 lb. bags; 250-440 lb. drums; solution.	Iron, plastics, rubber, steel	50-60	Highly soluble	70-80% $Na_2Al_3O_4$, 32% $Na_2Al_3O_4$ minimum	Hopper agitation required for dry feed
Ferrous sulfate $FeSO_4 \cdot 7H_2O$	Copperas, green vitriol	Coagulation	Bags, barrels, bulk	Asphalt, concrete, lead, tin, wood	63-66	0.5 at 32°F 1.0 at 68°F 1.4 at 86°F	55% $FeSO_4$ 20% Fe	Hygroscopic; cakes in storage; optimum pH is 8.5-11.0
Ferric chloride $FeCl_3$ (37-47% solution)	Ferrichlor, chloride of iron	Coagulation	5-13 gal. carboys, trucks, tankcars	Glass, stoneware, synthetic resins	(11.2-12.4 lbs /gal)	-	37-47% $FeCl_3$ 13-16% Fe	Very corrosive
Ferric sulfate $Fe_2(SO_4)_3 \cdot 9H_2O$	Ferrifloc, Ferrisul	Coagulation	50-175 lb. bags, 200-425 lb. drums	Ceramics, lead, plastic, rubber, 18-8 stainless steel	56-72	Soluble in 2-4 parts water	66% Fe_2SO_4 20% Fe	Mildly hygroscopic, Coagulant at pH 3.5-11.0
Sodium silicate Na_2OSiO_2	Water glass	pH control	Drums, bulk (tank trucks, tank cars)	Cast Iron, rubber, steel	(11.6 lbs/gal)	Highly soluble	28.7% SiO_2	Variable ratio of Na_2O to SiO_2. pH of 1% solution is 12.3
Chlorine	Chlorine gas, liquid chlorine	Disinfection	100-150 lb. cylinders, 1 ton containers, 16-30-55-85 and 90 ton tank cars, tank trucks (about 15-16 tons)	Dry - black iron, copper, steel. Wet gas - glass, hard rubber, silver, earthenware	91.7 (liquid at 32°F)	0.063 at 50°F 0.047 at 86°F	99.8% Cl_2	Toxic gas. Solutions in water highly acidic and corrosive.

Table 5-1. Principal Chemicals Used in Water Treatment (Continued)

Chemical Name and Formula	Common or Trade Name	Purpose	Shipping Containers	Suitable Handling Materials	Bulk Density lb/cu ft	Solubility lb/gal	Commercial Strength	Characteristics
Calcium hypochlorite Approx. $Ca(OCl)_2 \cdot 4H_2O$	"HTH", "Perchloron", "Pittchlor"	Disinfection	5 lb. cans, 100-300-800 lb. drums	Glass, rubber, stoneware, wood	48	Approx. one	70% "available" chlorine	
Potassium permanganate $KMnO_4$	Purple salt	Taste-odor control	Bulk, barrels, drums	Iron, steel, plastics	90-100	0.5 at 70°F	98%	Danger of explosion on contact with organic matter
Activated carbon (powdered)	Powdered activated carbon	Taste-odor control	Bags, bulk	Dry-Iron, steel, Wet-rubber, silicon stainless steel	8-28	Insoluble (used as a 1 lb/gal slurry)	-	-
Activated carbon (granular)	Granular activated carbon	Taste-odor control	Bags, bulk	Dry-Iron steel, Wet-rubber, silicon iron, stainless steel	22-36	Insoluble	-	-
Calcium oxide CaO	Quicklime, burnt lime, chemical lime, unslaked lime	pH control	80 lb. bags; 100 lb. barrels, bulk (carloads or trucks)	Asphalt, cement, iron, rubber, steel	55-60	Slakes to form hydrated lime	75-99% CaO	pH of saturated solution is 12.4
Carbon dioxide CO_2	Carbon dioxide	ph control	20-50 lb. cylinders, 10-20 or 18-20 ton tank trucks, 30-43 ton tank cars	Dry-iron, steel Wet-rubber, ceramics	63.7 at 0°F (liquid)	0.03 at 32°F 0.014 at 68°F 0.008 at 104°F	99.9% CO_2	pH of saturated solution at 68°F is approx. 4.0.
Sodium chloride NaCl	Common salt, salt	Sodium zeolite regeneration	Bags, barrels, bulk (carloads)	Bronze, cement, rubber	Rock 50-60 Fine 58-70	2.9 at 32°F 3.0 at 68°F 3.0 at 86°F	96% NaCl	Corrosive when moist.
Sodium hexametaphosphate	Polyphosphate, glassy phosphate, vitreous phosphate	Corrosion inhibitor	100 lb. bags, 100-320 drums	Hard rubber, plastics, stainless steel	Crystal 78 Flake 81 Powder 64	Highly soluble	67% P_2O_5	pH of 1% solution is 6.7-7.2

CHAPTER 6

WATER TREATMENT PLANT WASTES

6-1. Quantities and characteristics of wastes. In connection with water treatment plant location and design, the disposal of the wastes generated during the various treatment processes must receive consideration. Among these wastes are sludge from presedimentation basins, coagulation sludge, filter wash water, spent regenerant and rinse water from ion-exchange softeners, and mineral wastes from desalination facilities.

6-2. Waste management.

a. Water-treatment sludges.

(1) Presedimentation sludge. Presedimentation sludge may be disposed of by returning it to the stream from which the raw water was taken, provided the sludge is deposited in such a manner that it will not impede flow. The sludge could be dewatered in lagoons or sludge drying beds and then hauled to landfills or spread on land as an alternative.

(2) Coagulation sludge.

(a) Lagoons. If land is available near the treatment plant, alum sludge can be placed in lagoons to affect further concentration of solids. Depending on the local climate and the properties of the sludge, the final solids content in the lagoon may be as low as 1 percent or as high as 17.5 percent. Water should be removed from the lagoon by decantation. The decanted water may be returned to natural watercourses if deemed safe, and is sometimes returned to the treatment plant for recycling. Lagoons should be of sufficient size to contain the sludge generated per day for 1,825 days (5 years), plus precipitation less any decantation, evaporation, disposed sludge, or other losses. At least two lagoons should be provided so that fresh alum sludge can be placed in one while the alum sludge in the other is allowed to concentrate. After sufficient drying, the sludge should be removed from the lagoon and placed in a landfill or spread on suitable ground. The minimum solids content which should be attained before alum sludge can be removed from lagoons is generally about 10 percent. In colder climates, the freeze-thaw cycles to which the liquid in the lagoon is subjected will aid materially in concentrating the solids. Sludge lagoons should be enclosed by adequate fencing to exclude children and animals.

(b) Discharge to sanitary sewers. Alum sludges may also be discharged to sanitary sewers if disruption of wastewater treatment processes is not anticipated. If this procedure is chosen, precautions must be taken to insure that the sludge does not create a hydraulic

overload in the sewers or form significant deposits in the sanitary sewer. Inasmuch as a large portion of alum sludge is not biodegradable, the addition of alum sludge to wastewater will increase the sludge production at the wastewater treatment plant. Disposal of alum sludge to storm sewers is equivalent to disposal in natural watercourses and should not be attempted.

(c) Sludge beds. Another method of dewatering is application of the sludge to special sludge beds. These beds are usually composed of 6 to 12 inches of sand ranging in size up to 0.5 millimeter, with an underdrain system of graded gravel 6 to 12 inches deep. Drain pipes 6 to 8 inches in diameter are placed in the gravel to carry away the water from the beds. Sand beds can usually achieve a 20 percent solids concentration in alum sludge within 100 hours at a loading rate of 0.8 psf. However, the results are highly dependent on the characteristics of the sludge and local climatic conditions. Warm, dry climates are best suited to the use of sand drying beds. The water passing into the drain pipes should be suitable for disposal into natural watercourses. The dewatered sludge is usually removed from the sand bed by mechanical means, but a minimum solids content of approximately 20 percent must be attained before mechanical handling is practical. After removal, the dewatered sludge is usually hauled to a landfill.

(d) Mechanical dewatering devices. Several mechanical devices have been used for dewatering of alum sludge, including pressure filters, centrifuges, freeze-thaw devices, vacuum filters, and dual-cell gravity solids concentrators. Two or more of these processes can be used within the same system to obtain a higher degree of solids concentration than would be attainable using only one process.

b. Filter wash water. Wash water volumes range from about 1 to 3 percent of the raw water processed. Disposal of filter wash water may be by discharge to natural receiving waters, by recovery and reuse of the wash water, by lagooning, or by discharge to a sanitary sewer.

(1) Discharge to natural receiving waters. This means of disposal may be practiced if it does not affect other water users adversely.

(2) Recovery and reuse. Recovery and reuse are accomplished by mixing the filter wash water with the influent raw water before or at the rapid-mix basin. In most cases, the wash water is collected in a recovery basin from which it is pumped into the plant raw water inflow. Suspended solids in the wash water settle along with other solids in the plant basins and the only wastewater discharged from the plant is that associated with basin sludge removal. The recycling of filter wash water serves as a water conservation technique and may have economic advantages over other means of disposal. In some instances, the suspended particles in the filter wash water may not settle out easily, and recycling may, under some circumstances, cause abbreviated

filter runs. Another potential drawback of wash water recycling, particularly if the raw water has a high plankton count, is a build-up of algae in the recycled suspended matter and consequent increase of taste and odor in the water.

(3) Lagooning. Lagooning is an accepted means of managing filter wash water flows. If a separate lagoon is used for the wash water, the supernatant from the lagoon may be recycled through the water treatment plant.

(4) Discharge to sanitary sewer. Filter wash water may also be discharged to a sanitary sewer. Rate of flow regulation generally will be required to avoid sewer surcharge. This mode of disposal is most applicable if the characteristics of the wash water make it unsuitable for recycling.

c. Waste brines. Two types of brine flows can be generated at water treatment plants, regeneration brines from cation-exchange softeners and waste brines from desalination processes. These brines are very similar as far as disposal techniques are concerned. Methods of brine disposal include regulated discharge to surface waters, "evaporation" pond disposal, and discharge to a sanitary sewer. If pond disposal is utilized, the ponds must be lined to prevent seepage of brine into the ground water. Depending on the location of the water treatment plant and the volume of brine generated, these methods may vary widely in cost, reliability, and environmental acceptability.

(1) Discharge to surface waters. Unregulated discharge to surface waters is usually unacceptable. An exception is that waste brines from a desalting plant near the ocean can probably be discharged to the ocean if precautions are taken in the design of the outfall to insure that the brine is adequately diluted. On large rivers, it may be possible to store wastes in watertight ponds during low-flow periods and release them at a controlled rate during high flows. This may be an acceptable procedure if it can be shown that the wastes do not significantly affect water quality when released during the high-flow period.

(2) Evaporation ponds. Evaporation ponds can be used for disposal of waste brines if evaporation rates are high, precipitation is minimal, and land costs are low. In most localities, precautions must be taken to insure that brine ponds do not overflow or leak into the ground water. Watertight ponds are required for most situations.

(3) Discharge to a sanitary sewer. Disposal by regulated discharge to a sanitary sewer may be practiced if wastewater treatment plant operating personnel and regulatory authorities approve. Conventional wastewater treatment processes do nothing to remove dissolved minerals from water. Hence, all of the dissolved salts

discharged to the sanitary sewer will eventually be present in the effluent from the wastewater treatment plant.

APPENDIX A

DESIGN EXAMPLES

A-1. Clarification. This design example is based on the following conditions: river water source; no softening required; turbidity of raw water is variable, but rarely exceeds 1,000 units. See plant flow schematic, figure A-1, showing two-stage clarification treatment.

a. Facility to be served. The water treatment plant will serve a permanent installation.

b. Population served.

- Resident 6,000
- Nonresident 1,800
- Effective population = $\frac{1,800}{3}$ + 6,000 = 6,600

c. System design capacity.

- Capacity factor: 1.42 (based on effective population)
- Design population = (1.42)(6,600) = 9,372
- System design capacity, based on population = (9,372)(150) = 1,405,800 gpd. Use 1.41 mgd = 980 gpm = 2.18 cfs
- Special design capacity for industrial processes, independently determined: 0.79 mgd
- Total system design capacity = 1.41 + 0.79 = 2.20 mgd
  = 1,530 gpm
  = 3.40 cfs

d. Preliminary treatment.

(1) Rack and strainers. Provide coarse rack with 3-inch clear opening followed by hydraulically cleaned basket strainers with 3/8 inch clear openings ahead of each pump. Strainers sized to provide velocity of less than 2 fps through 3/8 inch openings.

(2) Pumps. Provide three pumps rated at 1.10 mgd each. This gives firm pumping capacity of 2.20 mgd. Maximum capacity is 3.30 mgd.

(3) Meter. Provide Venturi-type flow meter in pipeline from intake works to treatment plant. Meter should be sized to cover expected flow range, minimum to maximum. Flow meter should be equipped with flushing lines and bayonet cleaning rods.

(4) Presedimentation basins. Not required.

(5) Aeration. Not required.

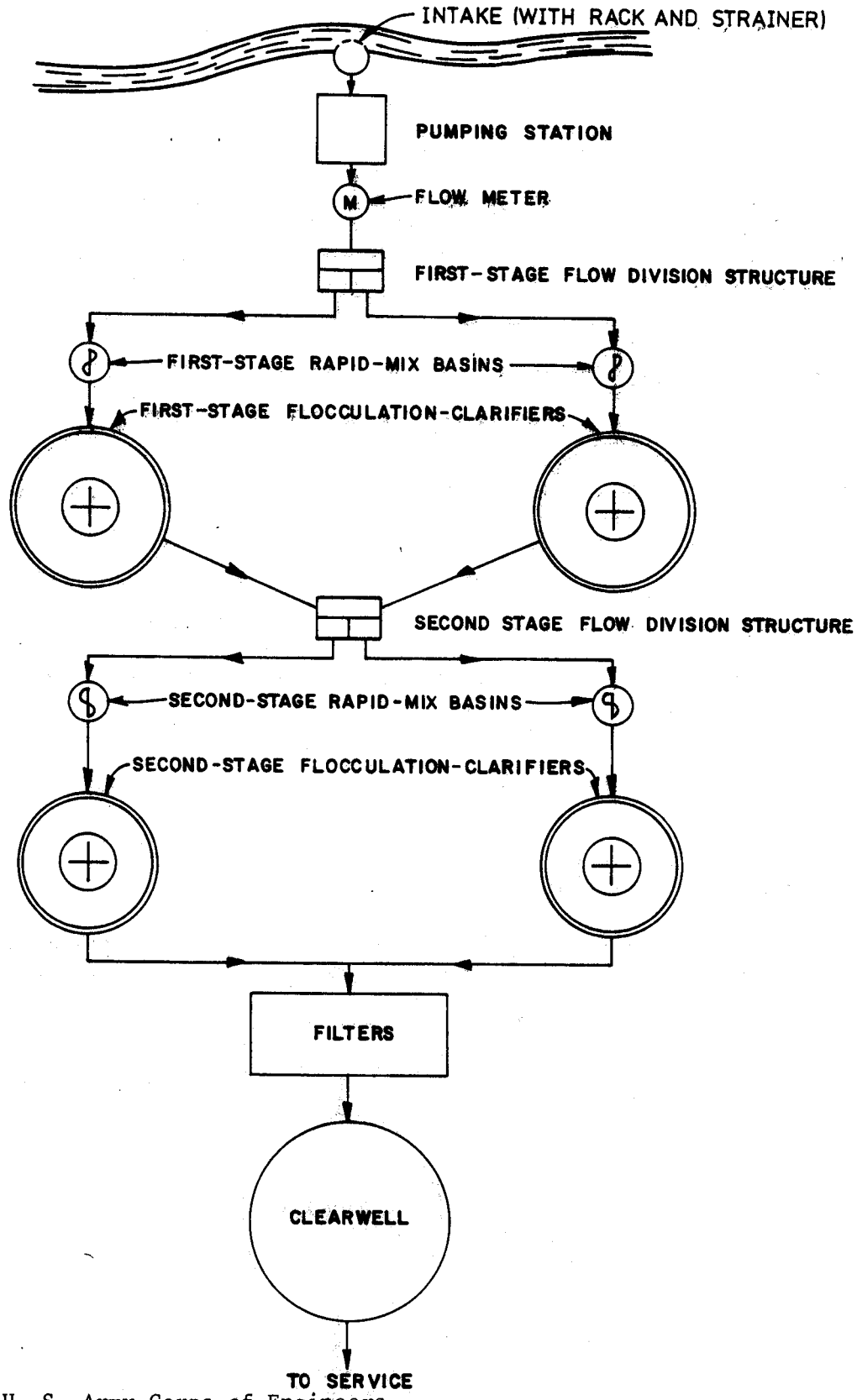

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FIGURE A-1. FLOW SCHEMATIC WATER CLARIFICATION PLANT

(6) Flow division. Normally, flow division is to be maintained through the second stage of treatment. Provide first-stage flow division structure with two identical rectangular weirs which will split flow into two equal parts. Hydraulic design of division structure should be such that flow, corresponding to maximum pumping capacity (3.3 mgd), can be carried through either half. Provide plates or gates so that either half of flow from division structure can be stopped.

e. First-stage mixing and sedimentation.

(1) Rapid mix. Provide two identical rapid-mix basins, each providing a detention time of 20 seconds at 50 percent of design flow. Volume of each basin is (20)(0.5)(3.4) or 34 cubic feet. Provide one electric motor-driven, rapid-mix unit in each basin which will yield a G value of approximately 700/second at a water temperature of 50 degrees F.

(2) Flocculation-sedimentation. Provide two mechanically-equipped, circular flocculator-clarifiers, each sized for 50 percent of design flow. These units will normally operate in parallel.

- Detention time in flocculation zone: 30 minutes.
- Detention time in peripheral sedimentation zone: 3 hours.
- Basin side-water depth: 12 feet.
- Flocculator depth: 10 feet.
- Flocculator volume = (30)(60)(0.5)(3.4) = 3,060 cubic feet.
- Flocculator area = 3,060/10 = 306 square feet.
- Flocculator diameter: 19.7 feet.
- Volume of sedimentation zone: (3)(60)(60)(0.5)(3.4) = 18,360 cubic feet.
- For 12 foot basin depth, sedimentation area = 1,530 square feet.
- Total basin area = 306 + 1,530 = 1,836 square feet.
- Upflow rate in sedimentation zone =

$$\frac{(.5)(2.2)(10^6)}{1,530} = 719 \text{ gpd/square foot or } 0.5 \text{ gpm/square foot, a satisfactory value}$$

- basin diameter = $\sqrt{\frac{1,836}{\pi/4}}$ = 48.3 feet

- Use double, V-notch, effluent weirs with inboard, effluent launder.

- Approximate total weir length, each basin: (48.3 - 4.3) (π) (2) - 51 = 225 feet. The 4.3 feet represents twice the distance of the weir from the tank wall. Fifty-one feet is the allowance made for non-effective weir length such as obstructions or supports.

- Weir loading = $\frac{(.5)(2.2)(10^6)}{225}$ = 4,890 gpd/feet

Flocculator units should have variable speed drives which will yield a G value of approximately 50/second at mid-speed and a water temperature of 50 degrees F. Provide piping and valves so that either basin can be bypassed while maintaining following secondary units in service. Provide basin overflow and basin cover structure if climatic conditions require. Provide basin sludge withdrawal piping to point of sludge disposal. Sludge pumps will be required if gravity sludge flow is not feasible.

f. Second-stage flow division structure. Provide second-stage flow division structure identical to first-stage. This structure allows combining the flows from the two, parallel-operating, first-stage sedimentation units followed by their division into two equal flows, which are then directed to the second-stage rapid-mix basins. Use of this flow-division structure provides maximum flexibility and optimum use of plant facilities when one first- or second-stage rapid mix or flocculator-clarifier is out of service for repair or maintenance.

g. Second stage mixing and sedimentation.

(1) Rapid mix. Provide two second-stage rapid-mix units identical to those used for first-stage rapid mixing.

(2) Flocculation-sedimentation. Provide two mechanically equipped, circular, flocculator-clarifiers identical in size to those used in the first-stage. These units will provide 30 minutes flocculation time and 3 hours sedimentation time. Total time for flocculation, both stages, is 60 minutes. Total sedimentation time, both stages, is 6 hours.

h. Filtration. Determine number of filter units.

- $N = 2.7\ (Q)^{1/2}$
- $Q = 2.2$ mgd
- $N = 2.7\ (2.2)^{1/2} = 4$ units

Filter configuration will consist of two filters, side by side, along both sides of a gallery sized to accommodate filter piping, valves, controls, etc. Main influent header pipe will be sized for a velocity not to exceed 1.5 fps. Calculated pipe diameter is 20.4 inches. Use 24-inch pipe giving actual velocity of 1.08 fps. Use 12-inch pipes to supply individual filters. At a rate of 2 gpm per square foot, total filtration area required will be 1,530/2 or 765 square feet or 191 square feet of medium area per filter. Provide 14 feet by 14 feet medium area. Use dual-media filters with 8 inches of filter-grade sand, 4 inches of coarse sand, and 20 inches of filter-grade anthracite, equipped with rotary surface wash capable of producing 0.5 gpm. Actual filter cells, including gullet, will be approximately 14 feet by 18 feet. Use vitrified clay tile or similar underdrains and gravel layer as recommended by manufacturer. Provide rate controllers and filter level control equipment. Establish overall depth of filter cell at 15 feet. Assume an arbitrary operating floor elevation of 15 feet. Significant filter elevations and related depths will be approximately as follows:

	Elevation (feet)
- Filter cell bottom	0.00
- Top of underdrains (plus 10 inches)	0.83
- Top of gravel (plus 10 inches)	1.67
- Top of coarse sand (plus 4 inches)	2.00
- Top of filter sand (plus 8 inches)	2.67
- Top of anthracite (plus 20 inches)	4.33
- Bottom of surface wash equipment (plus 2 inches)	4.50
- Bottom of troughs (plus 14 inches)	5.67
- Operating water level (7 feet above anthracite)	11.33

Depth of water above bottom of filter cell = 11.33-0.00 = 11.33 feet Freeboard = 15.00-11.33 = 3.67 feet. All four filters, each operated at 2 gpm per square foot, theoretically will produce

(4)(14)(14)(2)(1,440) or 2,257,920 gallons of water in 24 hours. Assuming two filters washed at 15 gpm per square foot for 15 minutes each in each 24-hour period, wash water requirements will be (2)(14)(14)(15)(15) or 88,200 gallons for normal backwash, or about 4 percent of production. Surface washing will require an additional (2)(14)(14)(0.5)(15) or 2,940 gallons. Total downtime, each filter, is assumed to be 20 minutes. The theoretical net water production for delivery to service will be as follows:

- Theoretical total production	2,257,920 gallons
- Loss due to filter down-time	15,680 gallons
- Less wash water required	91,140 gallons
- Net theoretical production available for service	2,151,100 gallons

Under actual operating conditions, with the filter-rate and level control equipment specified, the filters remaining in service will automatically compensate for production lost as a result of a filter being out of service for washing or repair. Level control insures that filter outflow will always match inflow. In addition to flow and level control equipment, provide automatic filter shut-off and alarm equipment to be activated at maximum allowable clearwell level and also provide filter high-level alarm. Provide all essential piping, valves, surface and backwash facilities, and operating consoles. Provide essential instrumentation for each filter. Provide sampling taps for each filter. Plant layout and hydraulic design should be such that additional filters can be readily added as required.

i. Clearwell. Clearwell (filtered water storage) capacity should be related to the available or proposed distribution system storage (ground and elevated). As an approximation for this design example, clearwell capacity of at least 0.6 million gallons could be provided. This is about 25 percent of the plant's daily production when operating at 2.2 mgd (design capacity). Greater clearwell capacity may prove advantageous depending on water demand patterns and plant operating schedule. The clearwell is commonly located adjacent to the filters and at an elevation permitting gravity flow to it. This usually requires an underground structure. An alternative arrangement consists of a sump following the filters, equipped with automatically controlled transfer pumps, which discharge to an above-ground tank or basin. Underground clearwells are commmonly constructed of reinforced concrete. For above-ground installations, steel tanks can be used. Regardless of the arrangement, the clearwell should be an independent structure, watertight, and protected against birds, animals, and insects. Vents must be installed and protected against surface and rainwater entry, birds, insects, and animals. A protected,

free-discharge, overflow should also be provided. The overflow must not be connected to any sewer or drain. Access to the interior of the clearwell is required. The access opening should be curbed at least 6 inches above the roof surface and be equipped with a hinged, overlapping, watertight, locking cover. As a general rule, the clearwell should be located at least 50 feet from sewers or drains. The area around the clearwell should be fenced and the site graded so that surface drainage is away from the structure. Where winters are severe, special consideration must be given to the design of vents and overflows to prevent freeze-up as a result of vapor condensation. A water level sensing instrument with readout in the plant control center should be provided. This can operate in conjunction with the previously described maximum level control-alarm circulation and lengthened chlorine contact time.

j. Hydraulic profile. The hydraulic interrelationship of major plant units must be carefully considered. In general, the hydraulic design of the plant should be on the side of ample flow capacity so that, under emergency conditions, water can be treated and filtered at considerably more than the normal rate with all filters in service. The approximate elevation data, given in the following tabulation, establish the emergency-operation hydraulic profile:

Location	Elevation (feet)
Water level upstream, first-stage flow division weir	100.00
Water level downstream, first-stage flow division weir	99.00
(Loss: raw water pipe friction + velocity head in pipe	0.42)
Water level in first-stage rapid-mix basin	98.58
(Loss: rapid-mix to first-stage flocculator-clarifier	0.50)
Water level in first-stage flocculator basin	98.08
(Loss: flocculator to sedimentation basin	0.01)
Water level in first-stage sedimentation basin	98.07
First-stage sedimentation basin emergency overflow level	98.57
(Loss: sedimentation basin weirs, launder and piping to second-stage division structure	1.50)
(Loss: second-stage division structure	1.00)
(Loss: division structure to second-stage rapid mix	0.50)

Location	Elevation (feet)
Water level in second-stage rapid-mix basin	95.07
(Loss: rapid-mix to second-stage flocculator clarifier	0.50)
Water level in second-stage flocculator	94.57
(Loss: flocculator to sedimentation basin	0.01)
Water level in second-stage sedimentation basin	94.56
Second-stage sedimentation basin emergency overflow level	95.06
(Loss: sedimentation basin weirs, launder and filter influent piping	1.80)
Water level in filters	92.76
Top of anthracite (7.00 feet below filter water level)	85.76
Filter operating floor level (3.67 feet above filter water level)	96.43
(Filter freeboard at second-stage basin overflow level	1.37)
Bottom of filter cells (15.00 feet below filter operating floor level)	81.43
(Maximum loss through filter media, gravel underdrains, effluent piping and controls	10.00)
Maximum water level in clearwell	82.76
(Overall head loss during emergency operation: first-stage division structure - maximum clearwell level	17.24)

By utilizing higher-than-normal chemical dosages, the plant can be operated under emergency overload conditions and still produce water meeting drinking water standards. In deriving the above data, the following emergency conditions were assumed:

(1) All raw water pumps are operated, giving a raw water flow of 3.3 mgd, which is 1.5 times nominal design rate.

(2) Both first- and second-stage rapid mix, flocculation, and sedimentation units on one side of plant are out of service.

(3) Reference elevation of water in first-stage flow division structure upstream from flow division weirs is arbitrarily established at the 100 foot mark.

(4) Raw water transmission pipe is assumed to be a flow division structure to first-stage rapid mix with a 44.5 foot, 16-inch pipe having a C value of 100.

(5) Plant units in service:

- Half of first-stage flow division structure.
- First-stage rapid mixer and first-stage flocculator-clarifier.
- Half of second-stage flow division structure.
- Second-stage rapid mixer and second-stage flocculator-clarifier.
- All (4) filters, with filter level control equipment, etc., operating normally.

(6) Filters are to be washed when head loss exceeds approximately 8 feet.

k. Wash water. Water supply for filter backwash can be supplied by a special pump, sized to provide the required flow. If used, backwash pumps should be provided in duplicate. An alternative is an elevated wash water storage tank providing gravity flow. The capacity of this tank should be at least 1.5 times maximum wash water requirement for a single filter. For this example, it is assumed that two filters will be washed in succession, each for 15 minutes, at maximum rate. A water tank having a capacity of three times the wash requirement for a single filter is recommended. Its capacity will be: (3)(15)(14)(14)(15) or 132,000 gallons. The wash water storage tank is refilled by pumping filtered water from the clearwell. Duplicate, automatically-controlled, refill pumps should be provided. A single pump should be capable of refilling the wash water tank in approximately 4 hours. A wash water rate-of-flow controller should be provided on the main wash water line serving the filters. Rate of wash water flow and totalizing instrumentation with readout visible during the washing process should also be provided.

l. Wash water recovery. Filter wash water can be recovered and recirculated through the plant. Solids contained in the wash water are removed in the plant sedimentation basins. Wash water recovery requires the construction of a basin into which the wash water is discharged. The basin bottom should slope steeply toward the suction pipe of the recycling pump. The capacity of the basin should be

approximately equal to the value of two, maximum rate, 15-minute filter washes, or about 90,000 gallons. For an assumed schedule of two filter washes every 12 hours, the recycle pump should have a capacity of about 125 gpm so that the recovery tank will be emptied in about 12 hours. The recycle pumps should be provided in duplicate. The recovery tank should be equipped with an overflow and a drain connection, both discharging to the plant waste disposal system. Under unusual circumstances, associated with raw water quality, it may be undesirable to recycle wash water. For such a situation, the wash water can be discharged to the recovery basin and then drained slowly to the plant waste disposal system.

m. Chemical application. Table A-1 summarizes major features of chemical application and related factors.

n. Chemical storage space. Chemical storage space requirements must be analyzed in terms of required application rates, shipping schedules and quantities. In general, a 30-day supply of a given chemical, based on estimated average feed rate, is the minimum storage volume that should be provided. If chemicals are purchased in bulk, the minimum storage volume available should be 150 percent of the volume of one bulk shipment, or about 30 days of storage at average feet rate, whichever is greater. For example, if the chemical purchase contract is for liquid alum, depending on local conditions, the manufacturer may elect to ship as follows: rail tank cars, 7,000 to 10,000 gallons; tank trucks, 3,600, 4,000 or 5,000 gallons. For rail delivery, minimum storage should be 1.5 x 10,000 or 15,000 gallons. For tank-truck delivery, minimum storage should be 1.5 x 5,000 or 7,500 gallons. If the estimated average alum feed rate is 30 mg/l and the plant is operated at design rate, 2.2 mgd, daily requirements, in terms of dry alum, are (30)(8.34)(2.2) or 550 pounds per day. Liquid alum, as furnished by the manufacturer, normally contains 5.4 pounds of dry alum per gallon of solution. The daily alum solution requirement will, therefore, be about 102 gallons. A storage volume of 15,000 gallons provides about 150 days of storage at average feed rate and design flow rate; a storage volume of 7,500 gallons, about 75 days. In this example, standard shipping volumes determine storage capacity. If the alum supply is to be purchased and stored in 100 pound bags, minimum bag storage space equivalent to (30)(550) or 16,500 pounds of alum should be provided. Loosely-packed, dry alum has a bulk density of about 50 pcf. The minimum bag storage volume should, therefore, be about 330 cubic feet arranged so that bags can be handled and stored on pallets. Suppliers should be consulted in advance of design regarding shipping quantities, schedules, and costs. It may be possible to reduce overall shipping and handling costs by providing plant storage capacity that will permit the manufacturer to ship in larger quantities.

A-2. Iron and manganese removal. Source of raw water - ground water, containing approximately 3.0 mg/l of iron and 0.25 mg/l of manganese.

No sulfide present. The pH of the raw well water is 7.5. Laboratory studies have indicated that iron and manganese oxidation and precipitation can be readily accomplished by aeration and chlorination, followed by a short period of flocculation. (Prediction of the performance of an iron-manganese removal treatment plant, based only on the chemical characteristics of the raw water, is difficult due to the variety of factors that can influence the removal processes. For this reason, laboratory and possibly pilot plant tests are desirable prior to the design of the treatment plant.)

a. Facility to be served. The water treatment plant will serve an Army boot camp.

b. Population served.

- Resident 5,200
- Nonresident 3,600
- Effective population = $5,200 + \frac{3,600}{3} = 6,400$

c. System design capacity.

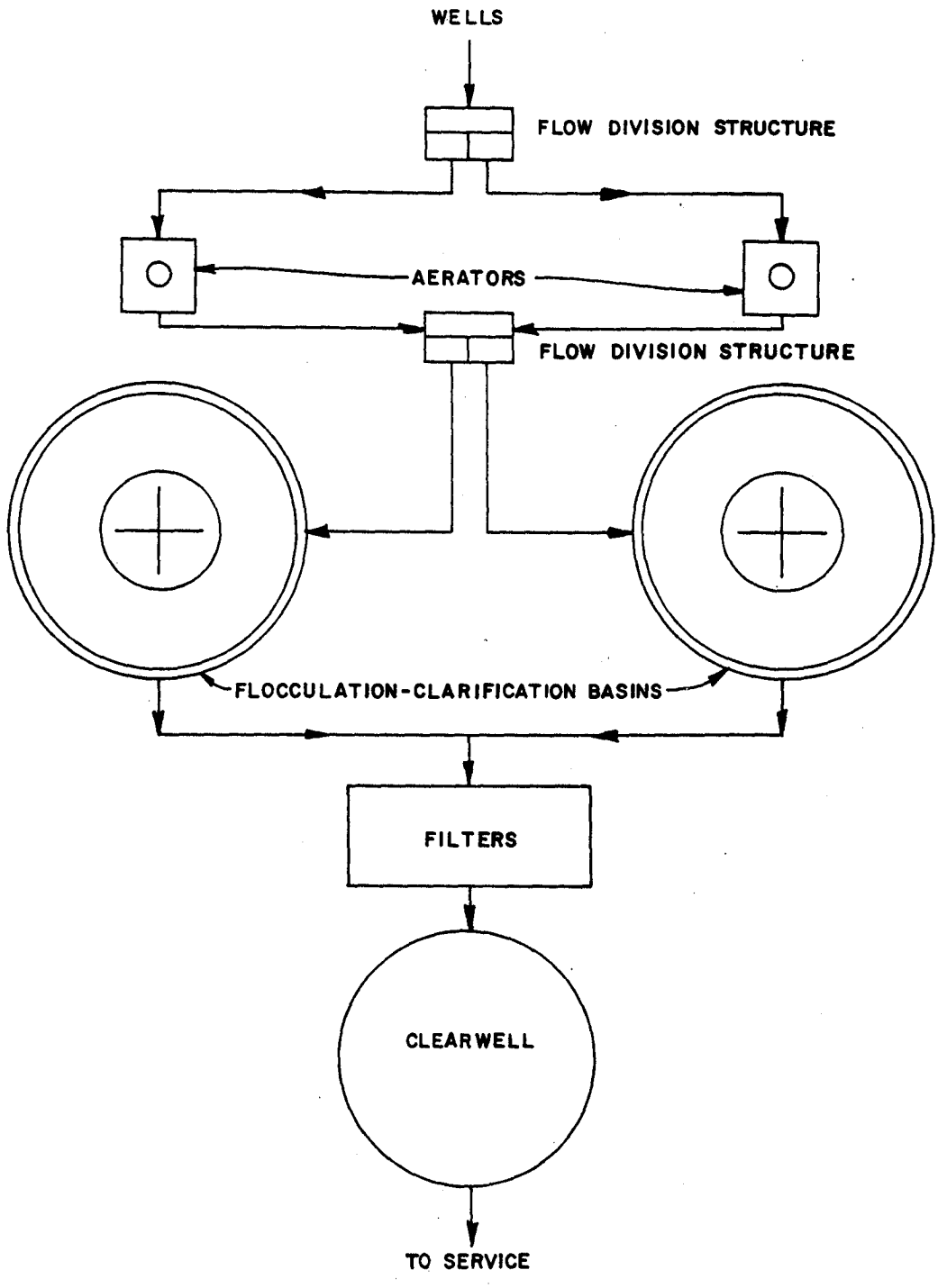
- Capacity factor: 1.43
- Design population = (1.43)(6,400) = 9,152
- System design capacity, based on population = (9,152)(150) = 1,372,800 gpd. Use 1.37 mgd
- Special design capacity for industrial processes and landscape irrigation: 0.30 mgd
- Total system design capacity: 1.37 + 0.30 = 1.67 mgd = 1,160 gpm = 2.58 cfs

d. Aeration. Provide flow-division structure followed by two natural-draft, multiple-tray aerators. Flow is divided equally between aerators. Each aerator has a distributor tray, three coke trays, and a collector tray, with the coke and collector trays spaced 18 inches apart. Dimensions of trays are 7.5 feet by 7.5 feet. Hydraulic loading is 10 gpm per square foot. Each tray is filled to a depth of 6 inches with 2 inch pieces of coke. The distributor tray orifices are sized and spaced to insure 2 inches of head on the orifices. In a mild climate, aerators can be located outdoors where good, natural ventilation will be obtained. If severe freezing conditions are likely, aerators must be housed and mechanical ventilation provided. Flow-division structure ahead of aerators must be designed so that

either aerator can be taken out of service for maintenance while plant continues to operate at full capacity. See figure A-2.

e. Chlorination. Provide two identical solution-type chlorinators, each capable of applying up to 5.0 mg/l of chlorine to the aerated water. For a 5 mg/l dosage, the total daily chlorine requirement will be (5)(8.34)(1.67) or 69.6 pounds of chlorine per day. Apply chlorine to effluents from aerators.

f. Flocculation-sedimentation. Provide flow division structure for aerated water and two identical, circular, flocculator-clarifiers equipped for mechanical flocculation and sludge removal. Flocculator zone, located in center of the basin, should have a detention time of approximately 30 minutes at 50 percent of design flow. Peripheral sedimentation zone should have a detention time of approximately 2 hours, and an upflow rate of approximately 0.75 gpm per square foot (1,080 gpd per square foot). These requirements can be met by a flocculator-clarifier basin with an overall diameter of 36 feet and a side water depth of 12 feet. Clarifier effluent should be collected by means of a weir and launder along the periphery of the basin. Aerator effluent division structure allows either basin to be out of service while utilizing both aerators, or both basins in service while only one aerator is operating. Provide basin overflow piping and sludge withdrawal piping. Provide cover structure for basins if severe freezing conditions are anticipated.



U. S. Army Corps of Engineers

FIGURE A-2. SCHEMATIC OF IRON AND MANGANESE REMOVAL PLANT

Table A-1. Chemical Application

Chemical	Storage	Feed Equipment	Feed Points
Alum[1]	Chemical storage area	Dry	First- and second- stage rapid-mix
	Elevated hopper in chemical feeder room	Dry	
	Tanks in bldg.	Solution	
Ferric sulfate[1]	Chemical storage area	Dry	First- and second-stage rapid-mix basins.
	Elevated hopper in chemical feed room	Dry	
Poly-electro-lyte[2]	Chemical storage area	Solution	First-stage rapid-mix basins or Combined filter influent
Chlorine[4]	Separate ventilated room in chemical feed area	Solution[3]	Rapid-mix basins and/or Combined filter influent and/or Combined filter effluent
Potassium perman-ganate	Chemical storage area	Solution	First-stage rapid-mix basins
Activated carbon[5]	Chemical storage area area	Dry	First-stage rapid-mix basins and Combined filter influent

Notes:

[1] Alum (or ferric sulfate), the principal coagulant, and associated feed systems are critical to plant performance and water safety. Therefore standby feeding facilities and ample reserve coagulant storage must be provided.

[2] Alternative feed points are provided in the interest of treatment flexibility. Normally, polyelectrolyte will be applied at the first-stage rapid-mix basins; but under some raw water quality conditions, it may provide advantageous to feed an additional small quantity, 0.1 mg/l or less, of polyelectrolyte to the combined filter influent. Polyelectrolyte, so applied, acts as a filter aid and improves turbidity removal by the filtration process.

[3] Chlorine and the chlorine feed systems are critical to the production of safe water. Standby chlorination equipment and ample reserve chlorine storage space must be provided.

[4] Alternative feed points are provided in the interest of treatment flexibility. Normally, sufficient chlorine will be fed at the second-stage rapid-mix basins to provide a free chlorine residual in the effluent from the filters. The residual is then adjusted upward as necessary by application of a small amount of chlorine to the combined filter effluent. Chlorine will be applied to the combined filter influent only when little or no chlorine residual is present in the sedimentation basin effluents.

[5] Alternative feed points are provided in the interest of treatment flexibility. Activated carbon will be used on an intermittent basis, as required, for taste-and-odor reduction. When it is necessary to apply large quantities of carbon, it should be applied to the first-stage rapid-mix basins. For some tast-and-odor conditions, the effectiveness of the carbon may be increased by splitting the carbon dosage between the first-stage rapid-mix basins and the combined filter influent. When using a split carbon feed, the bulk of the carbon should be applied to the rapid-mix basins with no more than about 10 percent of the total carbon application going to the combined filter influent. Under these conditions, it probably will be necessary to increase the chlorine dosage to the combined filter effluent in order to provide disinfection and an adequate residual in the water delivered to service.

APPENDIX B

LABORATORIES AND LABORATORY ANALYSES

B-1. Minimum analyses for Army water treatment plants. The minimum number and frequency of analyses to insure drinking water of acceptable quality are determined by the size of the system and the treatment required. The frequency of analyses must also be adjusted locally to meet changing raw water characteristics. For purposes of establishing the required analytical frequency, water treatment plants have been divided into two classes, Class A and Class B. A Class A plant is any plant employing treatment beyond chlorination. A Class B plant is any plant which provides only chlorination. Minimum analysis frequencies are listed in table B-1.

Table B-1. Minimum Analysis Frequencies

PLANT CLASS A	PLANT CLASS B	MINIMUM ANALYTICAL FREQUENCY	CONSTITUENT/ANALYSES
X	X	2/day	Chlorine residual
X	X	2/day-plant, 2/week-system	Fluoride[1]
X	X	8/month for first 1,000 population. One additional sample/month for each 1,000 additional population.	Total Coliform[2]
X	X	2/month	Total Plate Count[2]
X	X	1/day	Turbidity (surface water)
X		1/day	Hardness[1]
X		1/day	Alkalinity[1]
X	X	2/day	pH
X		2/month	Temperature
X		2/month	Calcium
X		2/month	Total Filtrable Residue
X		1/week	Iron[1]
X		1/week	Manganese[1]
X		1/day to 2/month	Coagulation Test (where chemical addition is employed)

[1] Where controlled.

[2] The installation medical authority is responsible for bacteriological sampling of the potable water supply. However, in some instances (e.g., due to the remoteness of medical laboratory support), it may be desirable to have bacteriological surveillance capability available at the water treatment plant. Facilities engineering personnel should coordinate such instances with the installation medical authority to prevent needless duplication of equipment.

APPENDIX C

REFERENCES

Government Publications.

Department of the Army.

EM 1110-3-160	Water Supply, General Considerations.
EM 1110-3-161	Water Supply, Water Sources.
EM 1110-3-163	Water Supply, Water Storage.
EM 1110-3-164	Water Supply, Water Distribution Systems.
TB MED 576	Sanitary Control and Surveillance of Water Supplies at Fixed and Field Installations.

Nongovernment Publications.

American Water Works Association (AWWA), 6666 West Quincy Avenue, Denver, Colorado 80235

Manual.

M3	Safety Practice for Water Utilities.